APPENDIX D

SUMMARY OF INTERVIEWS WITH JAPANESE ENGINEERS

Agency	Date	Location	Participants
Japanese Engineers	May 31, 1994	Sheraton Washington Hotel, Washington D.C.	Japanese Engineers: Yashio Maekawa, Tomio Sekiguchi, Yasuo Kawauchi, Satoshi Kaburaki, Osamu Kuramoto, Osamu Nishino, Shigeru Watanabe, Nobuhiko Morita University of Nebraska-Lincoln: Jong H. Ock, Takashi Yamane

D.1. Criteria to determine decks to be replaced.

Japanese engineers generally use visual inspection as the first step to determine the significance of deterioration of bridge decks. They classify the status of deck deterioration into A, B, and C (A is the worst) according to their criteria in terms of crack, delamination, exposure, corrosion, and leakage and free lime. The following table shows their criteria in detail. In addition to the three classifications, they have one more category to indicate the degree of deterioration of bridge decks (referred to as "Circled A"). It represents a deck condition worse than level A, and in that case, the deck needs to be replaced as soon as possible.

Items	Level A	Level B	Level C
Cracks			
1. In two direction			
Crack Width	> 0.1 mm	> 0.1 mm	> 0.1 mm
Crack Spacing	< 40 cm	40 - 60 cm	> 60 cm
Crack Width	> 0.2 mm concentrated or polygonal	0.1 - 0.2 mm concentrated or polygonal	
2. In one direction			
Crack Width	> 0.2 mm	> 0.2 mm	0.1 - 0.2 mm
Crack spacing	< 50 cm	0.5 - 1 m	>1.0 m
Crack Width	> 0.2 mm concentrated	0.1 - 0.2 mm concentrated	
Delamination	> 0.3 sq. meter	0.3 - 0.1 sq. meter	< 0.1 sq. meter
Exposure and corrosion	Exposure length of main bar > 50 cm and Corrosion	30 - 50 cm and rusty	< 30 cm
Leakage and Free Lime	> 0.3 sq. meter and lime is observed	< 0.3 sq. meter and lime is observed	A little leakage and free lime

D.2. Methods in the replacement of bridge decks

(1) Japanese engineers said that they utilize various methods in replacing deteriorated bridge decks, that is, cast-in-place concrete with traditional formwork methods, full-depth precast panels, and stay-in-place forms (Concrete or Steel Metal Deck), in conjunction with project conditions and construction methods used in the initial deck construction. They mentioned that, if the initial bridge deck was constructed by means of cast-in-place concrete with conventional formwork, they generally use a cast-in-place method in replacing the bridge deck. If the initial deck utilized a precast panel system, they utilize a panel system in the replacement because of the difficulty in controlling camber in replacing cast-in-place concrete with a panel system.

(2) With respect to the replacement of bridge decks on steel girders, they said that they use non-composite designs for the existing bridges. Furthermore, they usually consider non-composite design in new bridge construction with steel girders and, in that case, they increase steel by 5% with precast planks. The following detail shows their current practice in the use of steel girders.

D.3. Night-time construction

They mentioned that since nighttime construction is more expensive than daytime operation, by a factor of approximately 1.5, they usually try to avoid nighttime construction.

APPENDIX E

BRIDGE DECK SYSTEM

Full-Depth Cast-in-Place Deck Systems

Full-Depth Cast-in-Place Deck System with Conventional Reinforcement

Design Assumptions and Procedure:

LRFD AASHTO Specification, 1st edition, strip design method.

Girder spacing	=	12.0 ft
Overhang length	=	4.0 ft

Specified concrete strength at time of opening the bridge for traffic = 4.0 ksi

Conventional reinforcement ASTM-A616, black steel:

yield strength	=	60 ksi
Top reinforcement clear cover	=	2.5 in.
Bottom reinforcement clear cover	=	1.0 in.

A 1/2 in.(12.5 mm) wearing surface is considered to be an integral part of the 9 in. slab

12 in. (300 mm) wide flange steel girders are considered as supporting elements of the slab

Design procedure is given in Appendix F. Fig. E.1 shows details of reinforcement for the test specimen. Dimensions of the test specimen were 20'-0" x 7'-6".

Construction: As the test specimens were constructed, time-logs were kept to record the time taken for each stage of construction. A local construction company was involved in construction of these specimens thus helping the research team evaluate the actual time taken to construct each test specimen. This information was intended to produce a comparison between the systems under laboratory construction constraints and for a specimen size of 20'-0"x 8'-0". Construction time recorded was as follows:

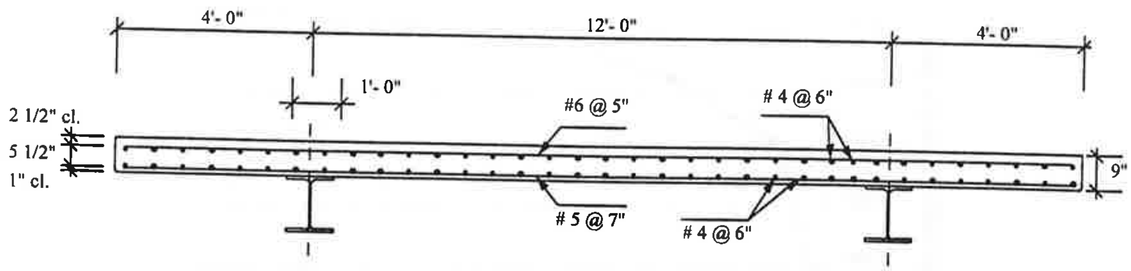


Fig. E.1 Full-Depth Cast-in-Place Deck System with Conventional Reinforcement

		Hours	Minutes
Construct forms (2 workers)		2.0	-
Reinforcement placing (2 workers)		1.0	30
Casting of concrete (3 workers)		1.0	-

Time / square ft = 4.5 hours / (20 ft x 7.5 ft) = 1.8 min./sq.ft.

Cylinders from concrete batches were taken and cured beside the test specimens to monitor the compression strength gain vs. time for the concrete. Cylinders were tested at 3, 7, 14, 28, and 90 days. Results are given in Fig. E.2. Actual concrete strengths exceeded the specified strength. Also, specimens from the conventional reinforcement were tested in tension. A stress-strain curve is shown in Fig. E.3.

Load testing: The testing program consisted of an ultimate test to failure. The objectives of the testing program were to determine the structural capacity of the system

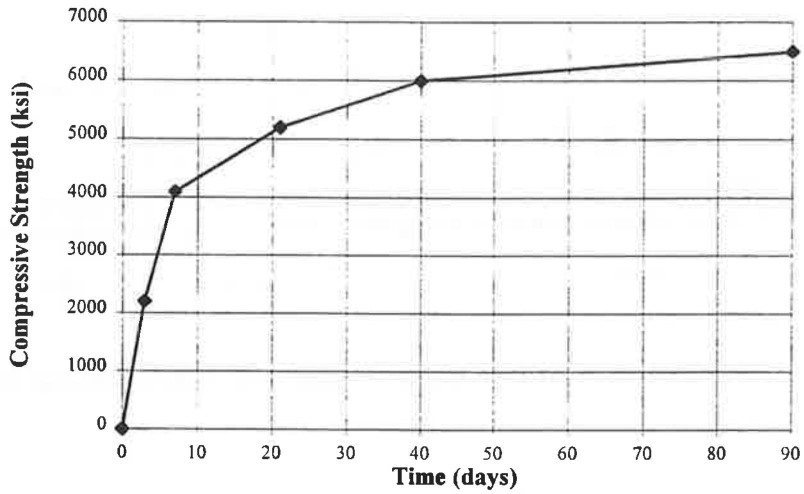


Fig. E.2 Concrete Compressive Strength vs. Time

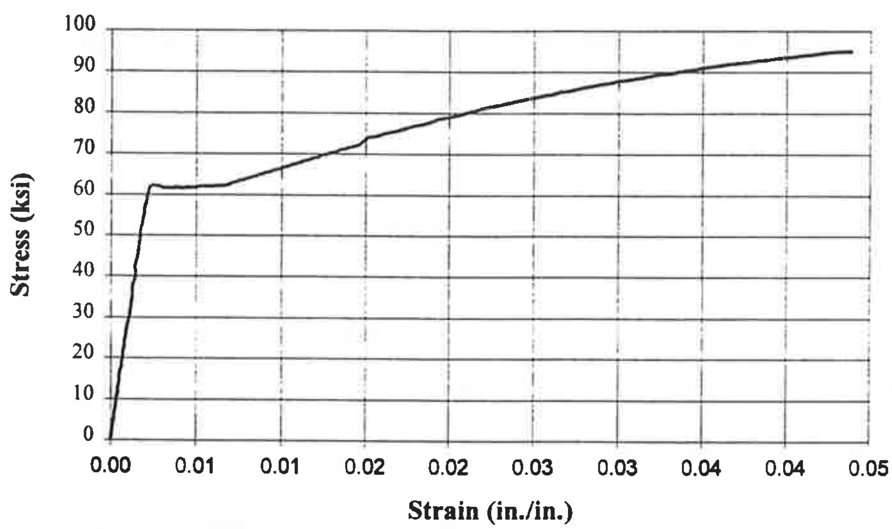


Fig. E.3 Stress-Strain Curve of Conventional Reinforcement

and the failure mechanism. The results of this test were considered as a base line for comparison with other cast-in-place systems.

Two simulated HS-25 AASHTO truck rear axle loads were used (an HS-25 loading utilizes an HS-20 AASHTO live load modified by a factor of 1.25). Fig. E.4 shows the test setup. The dimensions of the contact area of the point loads were 22.36 in. x 8.94 in. (568 mm x 227 mm). These dimensions were determined based on AASHTO Standard Specifications (*AASHTO Standard. 1995*), Art. 3.30, considering HS-25 AASHTO truck. AASHTO LRFD Specifications (*AASHTO LRFD. 1994*), Art. 3.6.1.2.5, gives different criteria to calculate the contact area dimensions. However, the tire contact area given by LRFD is almost the same as that given by AASHTO Standard if a load factor of 1.0 is used for Service I Limit State. Using the LRFD provision would lead to a 20 in. x 10.64 in. (508 mm x 270 mm) contact area. However, the research team decided to use the AASHTO Standard provision because the same test setup was intended to be used in testing other deck systems which were designed according to AASHTO Standard Specifications.

The load was applied over the center line of the specimen, as shown in Fig. E.5. To determine the structural capacity and failure behavior of the system, a monatomic ultimate load was applied at increments of 2 kips per second until failure occurred.

A series of strain gages was installed on the specimen before loading. Fig. E.6 shows the locations of the strain gages over the top and bottom surface of the specimen. The positions of the gages where chosen to cover the areas of maximum positive and negative moment. Displacement gages were also installed to measure displacement at the tip of the cantilever and at mid-span between girders as shown in Fig. E.6.

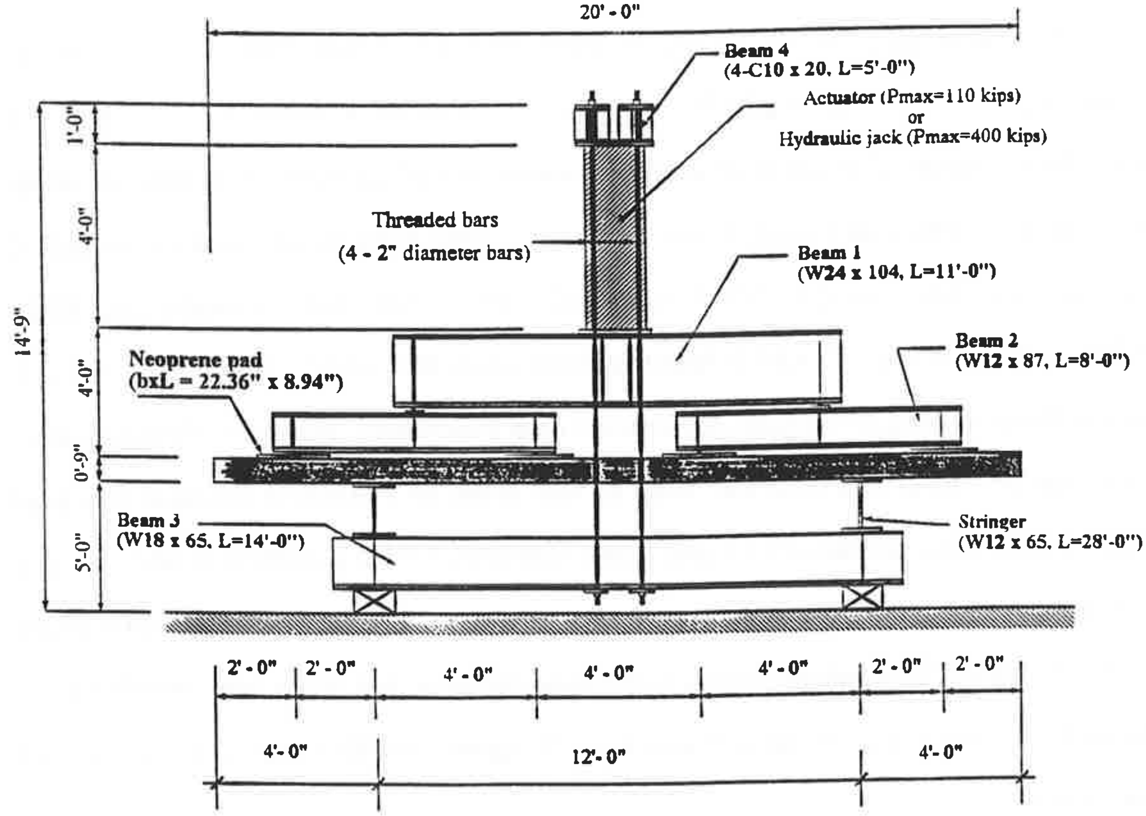


Fig. E.4 Test Setup

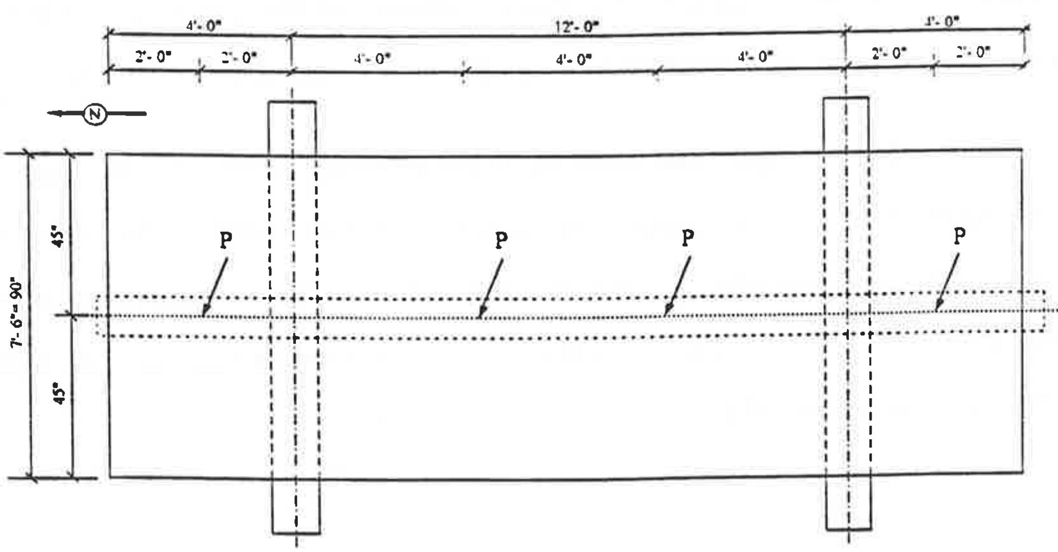


Fig. E.5 Load Positioning

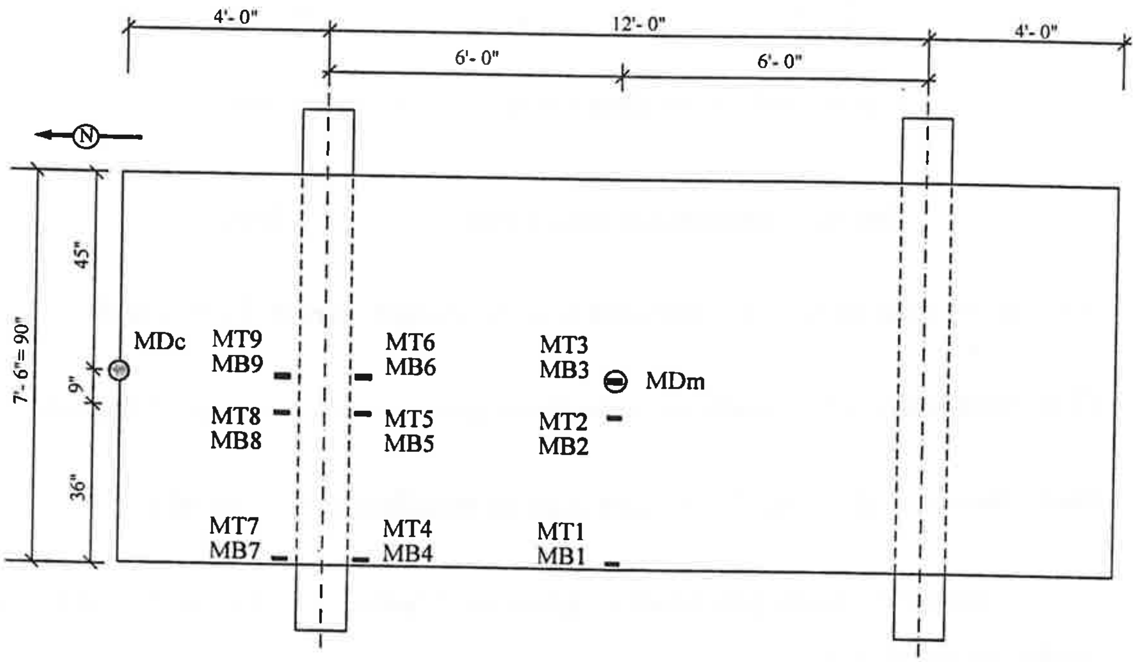


Fig. E.6 Strain and Displacement Gages Positions

(MT: top strain gages, MB: bottom strain gages, MD: displacement gages)

Full-Depth Cast-in-Place Deck System with Welded Wire Fabric

Design Assumptions and Procured:

AASHTO Standard Specifications, 15th edition.

Girder spacing	=	12.0 ft
Overhang length	=	4.0 ft

Specified concrete strength at time of opening the bridge for traffic = 4.0 ksi

Welded wire fabric reinforcement :

yield strength = 60 ksi

Top reinforcement clear cover = 2.5 in.

Bottom reinforcement clear cover = 1.0 in.

A 1/2 in. wearing surface is considered to be an integral part of the 9 in. slab.

12 in. wide flange steel girders are considered as supporting elements of the slab.

HS-25 loading (HS-20 AASHTO Live loading modified by a factor of 5/4).

Complete design procedure is given in Appendix F. Fig. E.7 shows details of reinforcement of the test specimen.

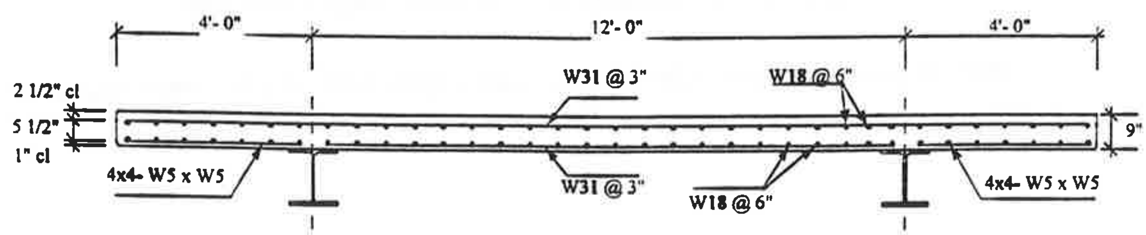


Fig. E.7 Full-Depth Cast-in-Place Deck System With Welded Wire Fabric

Note: positive reinforcement was not continuous over the girder line. This was done in order to study the effect of simplifying the reinforcement over the girder to help in rapid replacement of the deck. Due to epoxy coated welded wire fabric manufacturing limitations, the mesh, which is used for the negative moment area, is also used in the positive moment area. Also, instead of using W31@4 in., a W31@3 in. was used for the same reason.

Construction: The test specimen was constructed at the same time as the previous system under the same construction conditions. The construction time was as follows:

	Hours	Minutes
Construct forms (2 workers)	2.0	-
Welded wire fabric reinforcement placing (2 workers)	0	15
Casting of concrete (3 workers)	1.0	-

Time / square ft = 3.25 hours / (20 ft x 7.5 ft) = 1.3 min./sq.ft.

The same concrete batch used in the previous system, was also used in this system. Therefore, the results of testing for the concrete cylinders given in Fig. E.2 are valid here also.

Load testing: The test setup used in the previous system as shown in Figs. E.4 and E.5, was also used here. The testing program consisted of cyclic load testing and ultimate load testing. Initially, a service load of 25 kips (111 kN) per loading point, (simulating the rear wheel of HS-25 truck load plus impact), was applied monotonically to find the stress distribution over the test panel. This was followed by a cyclic load for two million cycles. The cyclic load range was from 5 kips (22.3 kN) to 25 kips (111 kN). Once the cyclic load test was completed, the monatomic service load was applied to compare the stress results with those before fatigue loading. Finally, to determine the structural capacity and failure behavior of the system, a monatomic ultimate load was applied until failure occurred.

A series of strain gages was installed on the panels before loading. Fig. E.8 shows the locations of the strain gages over the top and bottom surfaces of the panels.

Displacement gages were also installed to measure displacement at the tip of the cantilever and at mid-span between girders as shown in Fig. E.8.

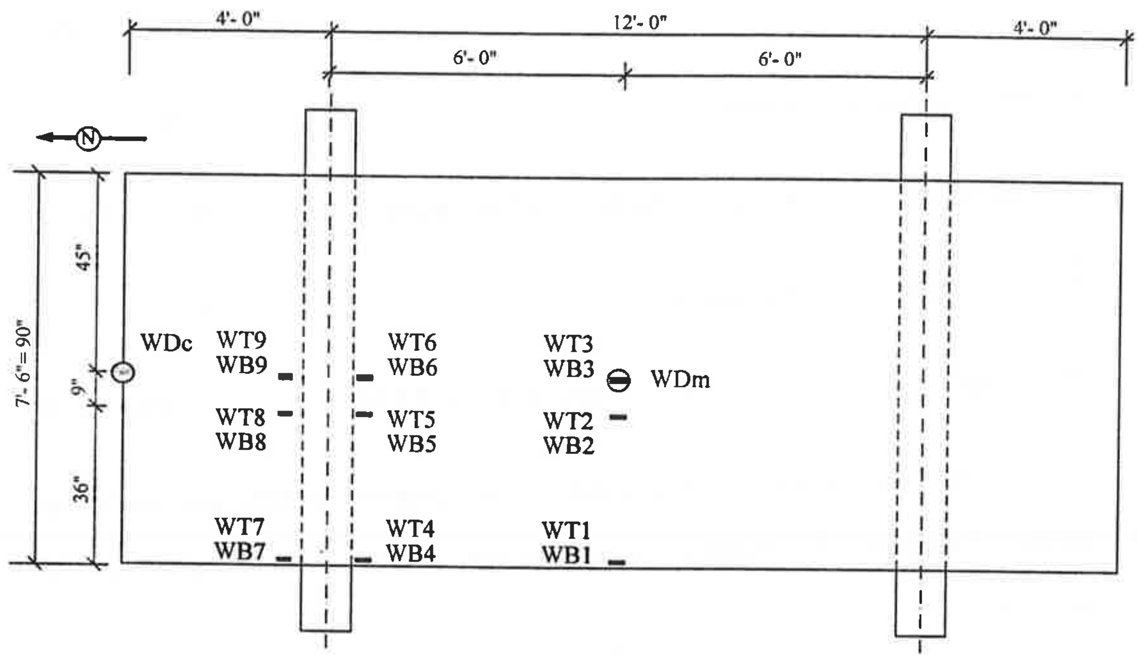


Fig. E.8 Strain and Displacement Gages Positions
(WT: top strain gages, WB: bottom strain gages, WD: displacement gages)

Precast Deck Sub-panel Systems

<u>*Conventional Precast Deck Sub-panel System*</u>

Design Assumptions and Procedures:

AASHTO Standard Specification, 15th edition.

Girder spacing = 12.0 ft

Overhang length		=	4.0 ft
Precast prestressed panel:	Concrete strength at release	=	4.0 ksi
	28-day concrete strength	=	10.0 ksi
	1/2 in. indented diameter strand, 270 ksi, low relaxation		
	Modulus of elasticity	=	29,000 ksi
	Panel dimensions:		
	(width x length x thickness)	=	(3'-9" x 11'-6" x 3")
Cast-in-place concrete:	Specified concrete strength at time of opening the bridge for traffic	=	4.0 ksi
	Reinforcement is welded wire fabric:		
	yield strength	=	60 ksi
	Modulus of elasticity	=	29,000 ksi
	Top reinforcement clear cover	=	2.5 in.
	Bottom reinforcement clear cover	=	1.0 in.

A 1/2 in.(12.5 mm) wearing surface is considered to be an integral part of the 9 in. slab.

12 in. (300 mm) wide flange steel girders are considered as supporting elements of the slab.

HS-25 loading (HS-20 AASHTO Live loading modified by a factor of 5/4)

Complete design procedure is given in Appendix F. Fig. E.9 shows details of reinforcement of the test specimen. Fig. E.10 shows a cross section of the precast prestressed panels.

Construction: Two 4 ft x 11.5 ft (1.2 m x 3.5 m) precast prestressed panels were cast in the prestressing bed available in the structural lab at the Omaha campus. Concrete cylinders were taken from the concrete batch, cured beside the panels, and tested

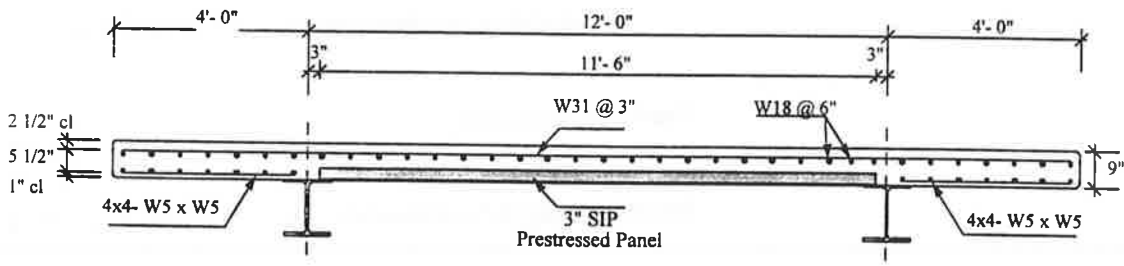


Fig. E.9 Precast Deck Sub-Panel System

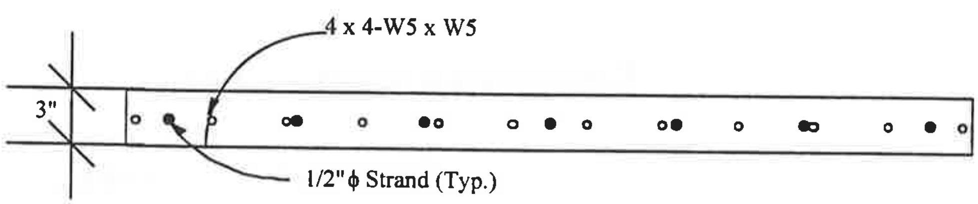


Fig. E.10 Cross Section of the Precast Prestressed Panel

at 3, 7, 14, 28, and 90 days to monitor the compression strength gain vs. time. Results are given in Fig. E.11. The concrete did not reach the specified 10,000 psi (69 Mpa) after 28 days.

The precast panels were erected between the steel girders and wood forming was used to form for the overhangs. The cast-in-place concrete topping was cast at the same

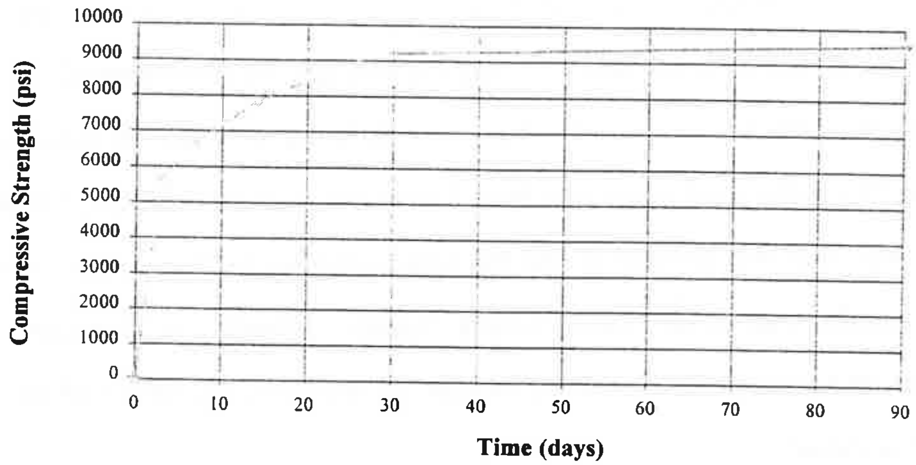


Fig. E.11 Concrete Compressive Strength vs. Time (Precast Prestressed Panel)

time that the full-depth cast-in-place deck systems were placed by a local contractor. A time-log was kept to record the time taken for each stage of construction with duration's as follows:

	Hours	Minutes
Prestressed panel placing*	0	10
Construct forms for the overhangs (2 workers)	1.0	-
Welded wire fabric reinforcement placing (2 workers)	0	10
Casting of concrete (3 workers)	1.0	-

* This time does not include time for producing the prestressed panels.

Time / square ft = 2.33 hours / (20 ft x 7.5 ft) = 0.93 min./sq.ft.

Load testing: The test setup used in the previous system as shown in Figs. E.4 and

E.5, was also used here. The testing program consisted of cyclic load testing and ultimate load testing. Initially, a service load of 25 kips (111 kN) per loading point, (simulating rear wheel of HS-25 truck load plus impact) was applied monotonically to find the stress distribution over the test panel. This was followed by a cyclic load for two million cycles. The cyclic load range was from 5 kips (22.3 kN) to 25 kips (111 kN). Once the cyclic load test was completed, the monotonic service load was applied to compare the stress results with those before fatigue loading. Finally, to determine the structural capacity and failure behavior of the system, a monotonic ultimate load was applied untill failure occurred.

A series of strain gages was installed on the panels before loading. Fig. E.12 shows the locations of the strain gages over the top and bottom surface of the panels. Displacement gages were also installed to measure the displacement at the tip of the cantilever and at the mid-span between girders as shown in Fig. E.12.

<u>*Continuous Precast Deck Sub-panel System*</u>

Design Assumptions and Procedures:

AASHTO Standard Specifications, 15th Edition,

Precast prestressed panel:

Concrete:	Specified release concrete strength	=	4.0	ksi
	Specified 28-day compressive strength	=	10.0	ksi

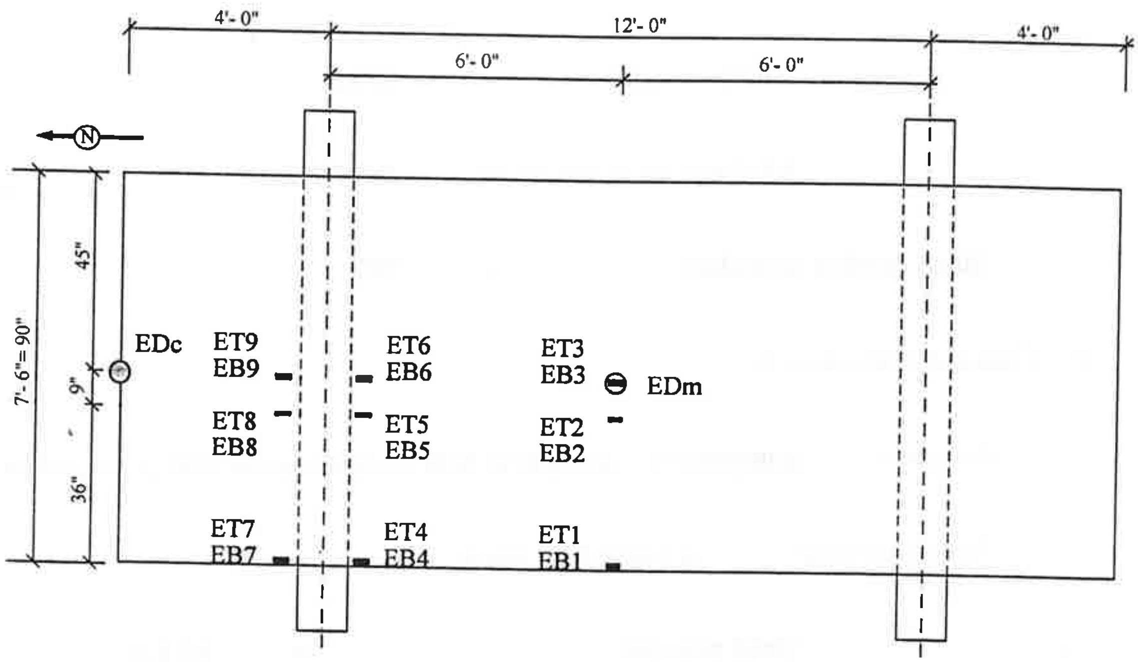


Fig. E.12 Strain and Displacement Gages Positions
(ET: top strain gages, EB: bottom strain gages, ED: displacement gages)

Prestressing reinforcement:

0.5 in. diameter indented strand, 270 ksi, low relaxation, ASTM-A421.

Ultimate strength	f'_s		= 270 ksi	
Yield strength	f^*_y	$= 0.9\, f'_s$	= 243 ksi	(Art. 9.15)
Initial prestressing	f_{si}	$= 0.75\, f'_s$	= 202.5 ksi	(Art. 9.15.1)
Modulus of elasticity	E_s		= 28,000 ksi	(Art. 9.16.2.1.2)

Non-prestressing reinforcement:

ASTM-A616, deformed bars, black steel

Yield strength = 60 ksi

Modulus of elasticity = 29,000 ksi (Art. 8.7.2)

Mean relative humidity = 70%

- Cast-in-place concrete:

Concrete: compressive strength at time of opening the bridge for traffic = 4.0 ksi

Reinforcement: welded wire fabric:

yield strength = 60 ksi

Modulus of elasticity = 29,000 ksi

Top reinforcement clear cover = 2.5 in.

Bottom reinforcement clear cover = N.A.

- Girder spacing = 12 ft

- HS-25 loading, equivalent to HS-20 AASHTO loading magnified with a factor of 1.25

- Open the bridge for traffic when the CIP deck slab has a compressive strength of 4000 ksi or more.

- 1/2 inch of the CIP topping was considered as an integral wearing surface.

- No loads should be applied on the overhangs of the SIP panels before the mortar mix filling the gaps over the girder lines achieves its specified strength of 4,000 psi.

- The SIP panel was designed to resist the loads of the topping slab self weight, a construction load of 50 lb./ft^2, and the deck finishing machine weight.

- The composite section was designed to resist the superimposed loads which are the future wearing surface self weight, barrier weight, and live load.

- A New-Jersey barrier type, of 330 lb./ft (4.82 N/mm) self weight, is considered.

For the design of the precast panel two stages were considered: (1) release of prestress; and (2) casting of the topping slab. At the release stage, compatibility and equilibrium equations were applied to the section at the gap over the girders to calculate the compressive stress gained in the #6 bars and the tensile stress lost in the prestressing strands. Therefore:

$$\varepsilon = \frac{A_p f_{pi}}{A_s E_s + A_p E_p} \qquad \text{(Eq. 3.1)}$$

where:

ε	= the elastic strain loss in the gap	
f_{pi}	= tensile stress in the strands just before release	= 0.75x270
		= 202.5 ksi (1396 MPa)
A_s	= the cross section area of the reinforcing bars	= 28x0.44
		= 12.32 in^2 (7948 mm^2)

A_p = the cross section area of the prestressing strands = 16x0.153

= 2.448 in^2 (1579 mm^2)

E_s = the Modulus of Elasticity in the reinforcing bars = 29,000 ksi (200x10^{3}MPa)

E_s = the Modulus of Elasticity in the prestressing strands= 28,000 ksi (193x10^{3}MPa)

$$\text{Therefore, } \varepsilon = \frac{2.448\text{x}202.5}{12.32\text{x}29{,}000 + 2.448\text{x}28{,}000} = 1.164\text{x}10^{-3} \text{ in./in.,}$$

compression stress in the reinforcing bars = ε (E_s)

= (1.164x10^{-3})(29,000) = 33.76 ksi (233 MPa)

and tensile stress in the prestressing strands = f_{pi} - ε (E_p)

= 202.5 - (1.164x10^{-3})(28,000) = 169.91 ksi (1171 MPa)

A similar analysis at the midspan between the girder lines was required to determine the tensile stress in the prestressing strands at that location. Calculations showed that this value was 191 ksi (1319 Mpa).

The reinforcing bars in the gap must be adequate to satisfy two design criteria: (1) preserve as much prestress in the strands as possible; and (2) transfer that prestress to the adjacent concrete without too much stress concentration. The first criteria was already covered in the preceeding paragraph. To satisfy the second criteria conservative approach was adopted by using the tension development length as the minimum required embedment into the concrete. This was considered to be conservative since the bars were expected to be predominantly in compression and the end bearing was totally ignored. However, the 18 in. (457 mm) embedment used was not considered wasteful in terms of the overall cost of the system. The buckling length of the #6 bars at the gap was also checked to protect these bars from buckling during handling or deck placing operations.

At the stage when the topping slab is cast, three sections were checked: (1) maximum positive moment section between the girders under construction loads, the self weight of the precast panel and the topping slab; (2) maximum negative moment section at interior supports under construction loads, the self weight of the precast panel and the topping slab; and (3) maximum negative moment section at the exterior support under the self weight of the precast panel and the topping slab, the construction load and the concentrated loads provided by the deck finishing machine. For the maximum positive moment section, the service concrete stresses and the ultimate flexural capacity of the precast panel were checked. For the maximum negative moment sections, the ultimate flexural capacity was checked. A complete design procedure of this system is given in Appendix F.

Construction: A sample 20 ft (6.1 m) wide bridge was constructed in the laboratory. The bridge consisted of two steel girders spaced at 12 ft (3.65 m) and two 4.0 ft (1.22 m) overhangs. The supporting steel girders had a 12 inches (305 mm) flange width. Fig. E.13 gives the cross section of the test specimen. Two 20 ft x 4 ft (6.1 m x 1.22 m) precast panels, P1 and P2, were cast as shown in Fig. E.14.

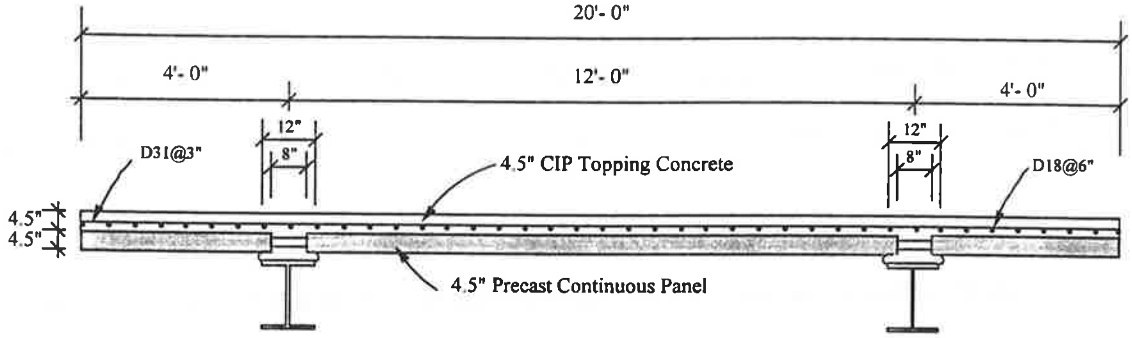


Fig. E.13 Cross Section of Test Specimen

The precast panels were produced using the prestressing bed facility in the structural lab or the Omaha campus. Concrete was produced by a local Ready Mix company. Concrete cylinders taken from the concrete batchs and cured beside test specimen were tested at 3, 7, 14, and 28 to monitor the strength gain vs. time. Figs. E.15 and E.16 show results of testing for the concrete cylinders.

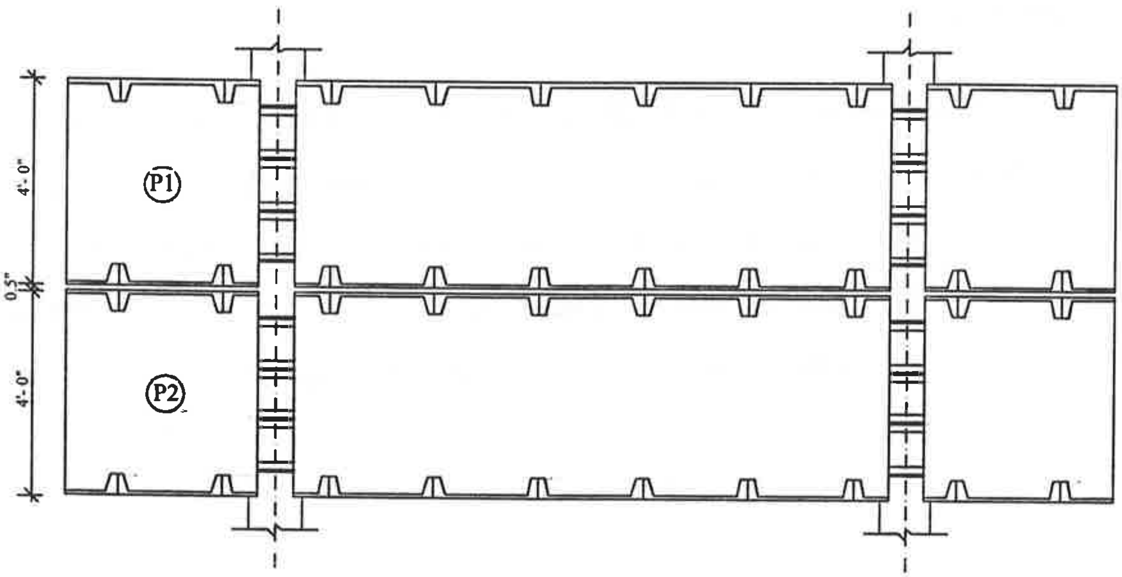


Fig. E.14 Plan View of Precast Panel

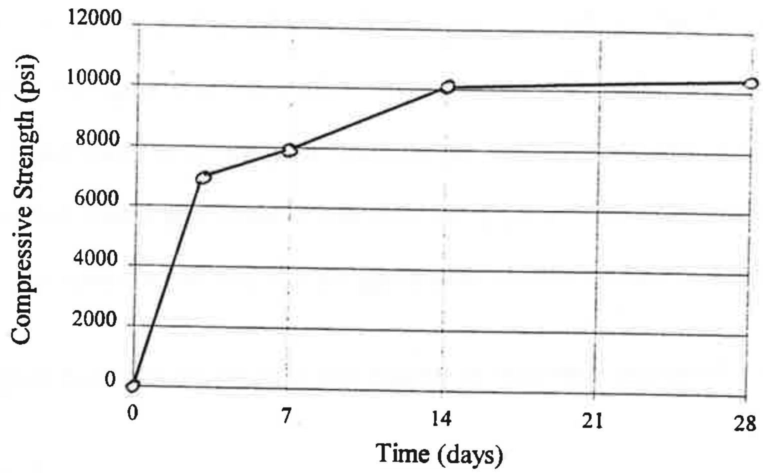


Fig. E.15 Concrete Compressive Strength vs. Time (Panel #1)

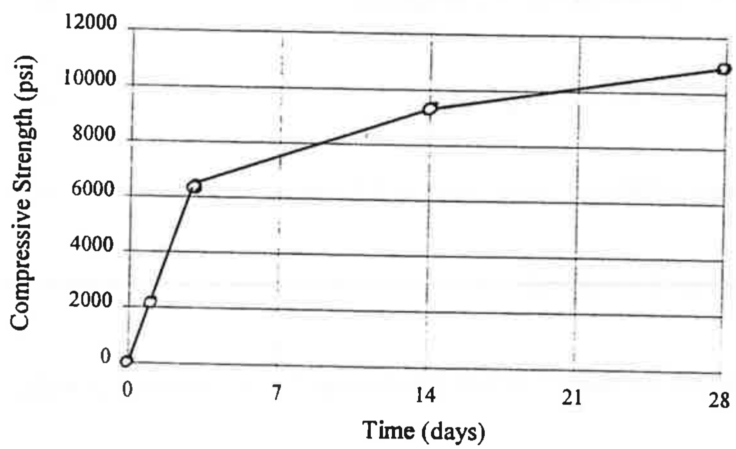


Fig. E.16 Concrete Compressive Strength vs. Time (Panel #2)

Once top surfaces of the steel girders were cleaned, the grout barriers were installed on the top flange and the first precast panel was set in its position. The spiral bars were compressed and installed in the pockets. Next, the second panel was butted to the first panel keeping 1/2 inch gap between them. The level of the precast panels was adjusted using the leveling devices. A flowable mortar mix of specified 4,000 psi (27.58

MPa) 28-day compressive strength was used to fill the 8 inch (203 mm) gaps over the girders. The #4 bars at the pockets were then spliced by placing a 9-inch (229-mm) long #4 bar and by releasing the spiral bars. Following this step the topping reinforcement, (welded wire fabric mesh), was set and the topping slab was cast. Concrete cylinders from the mortar mix and the topping slab were taken and cured beside the test specimen. Figs. E.17 and E.18 show results of testing for the concrete cylinders.

The following time-log was kept to record the time taken for each stage of construction:

	Hours	Minutes
Prestressed panel placing*	0	10
Filling the gaps with mortar mix (2 workers)	0	30
Splicing the prestressed precast panels	0	10
Welded wire fabric reinforcement placing (2 workers)	0	10
Casting of concrete (3 workers)	1.0	-

* This time does not include time for producing the prestressed panels.

Time / square ft = 2.0 hours / (20 ft x 8.0 ft) = 0.75 min./sq.ft.

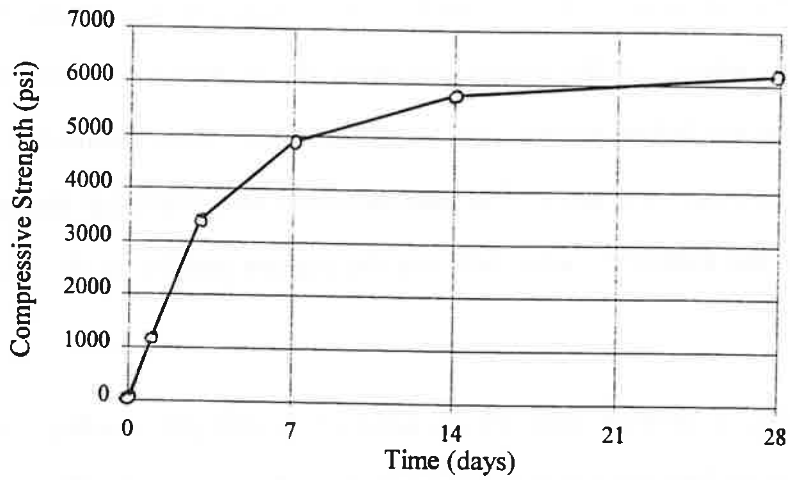


Fig. E.17 Concrete Compressive Strength vs. Time (Mortar Mix)

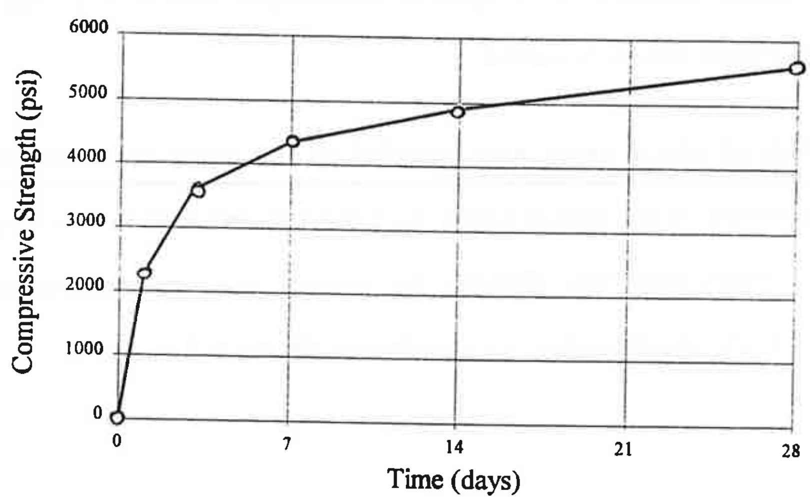


Fig. E.18 Concrete Compressive Strength vs. Time (Topping slab)

Loading Testing: The loading scheme consisted of an initial service load fatigue test followed by an ultimate test. The objectives of the loading scheme were to determine the structural capacity of the system, examine the system's behavior under cyclic load, and determine the failure machanism of each system. Two simulated AASHTO HS-25 truck rear axle loads were used. The load locations were placed at the center of the panel, directly over the transverse joint between the precast panels, as shown in Figs. E.4 and E.5.

Initially, a service load of 25 kips (111 kN) per loading point was applied monotonically to find the stress distribution over the test panel. This was followed by a cyclic load for two million cycles conducted while keeping the load intensity the same. Once the cyclic load test was completed, the monotonic service load was applied to compare the stress results with those before fatigue loading. To determine the structural capacity and failure behavior of the system, a monotonic load was applied in 10-kip (44.5 kN) increments until failure occurred.

A series of strain gages was installed on the panel before loading. Fig. E.19 shows the locations of the strain gages over the top and bottom surfaces of the panel. Displacement gages were also installed to measure the displacement at the tip of the cantilever and at the mid-span between girders as shown in Fig. E.19.

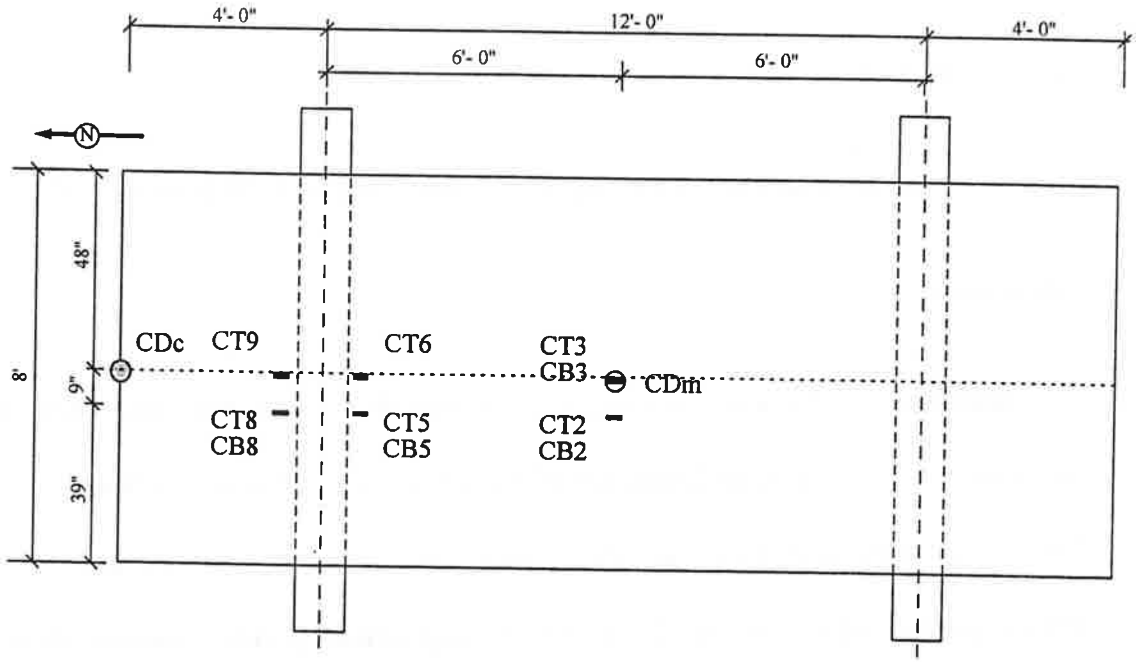


Fig. E.19 Strain and Displacement Gages

(CT: top strain gages, CB: bottom strain gages, CD: displacement gages)

Full-Depth Precast Deck System

Finite Element Analysis Results:

Results of Finite Element Analysis are shown in Figs E.20 through E.26.

Construction

Specimen Description and Testing Program Objective: A full-scale prototype was constructed at the structures laboratory of the University of Nebraska (Omaha Campus). The test specimen consisted primarily of three 20 ft long (6.0 m) and 8 ft (2.4 m) wide precast prestressed panels, two 26 ft (7.9 m) long girders, welded headless shear studs, welded threaded studs, non-shrink grout, and threaded bars for longitudinal post-tensioning.

The program and objectives of constructing this prototype consisted of the following :

- Precasting of concrete panels to confirm production feasibility.
- Constructing the bridge deck system to confirm feasibility and speed of construction.
- Experimental load testing to examine performance of the system including:

  - Serviceability of precast panels.

  - Fatigue resistance of precast panels.

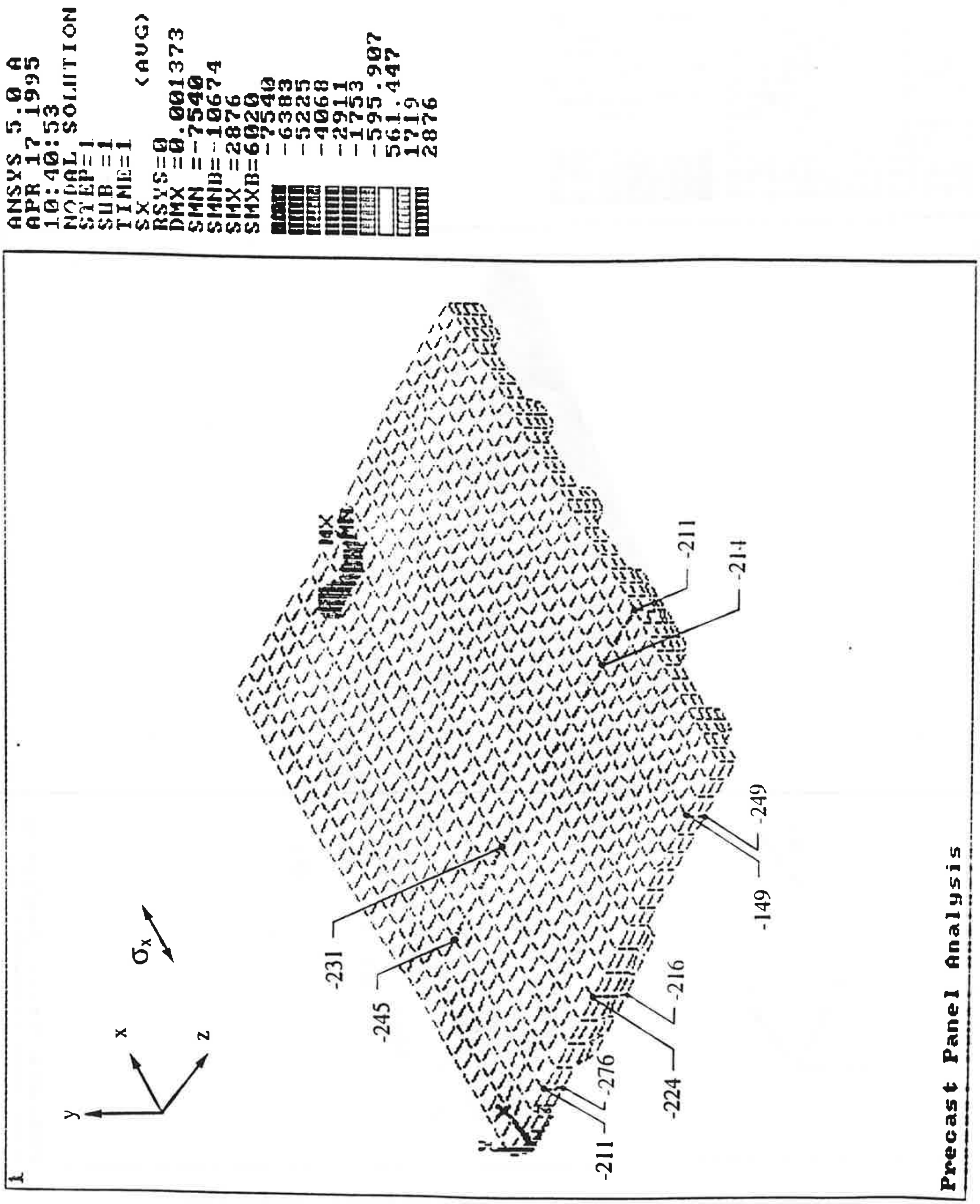


Fig. E.20 Stress in Transverse Direction due to Service Load (bottom stress)
(all stresses are in psi)

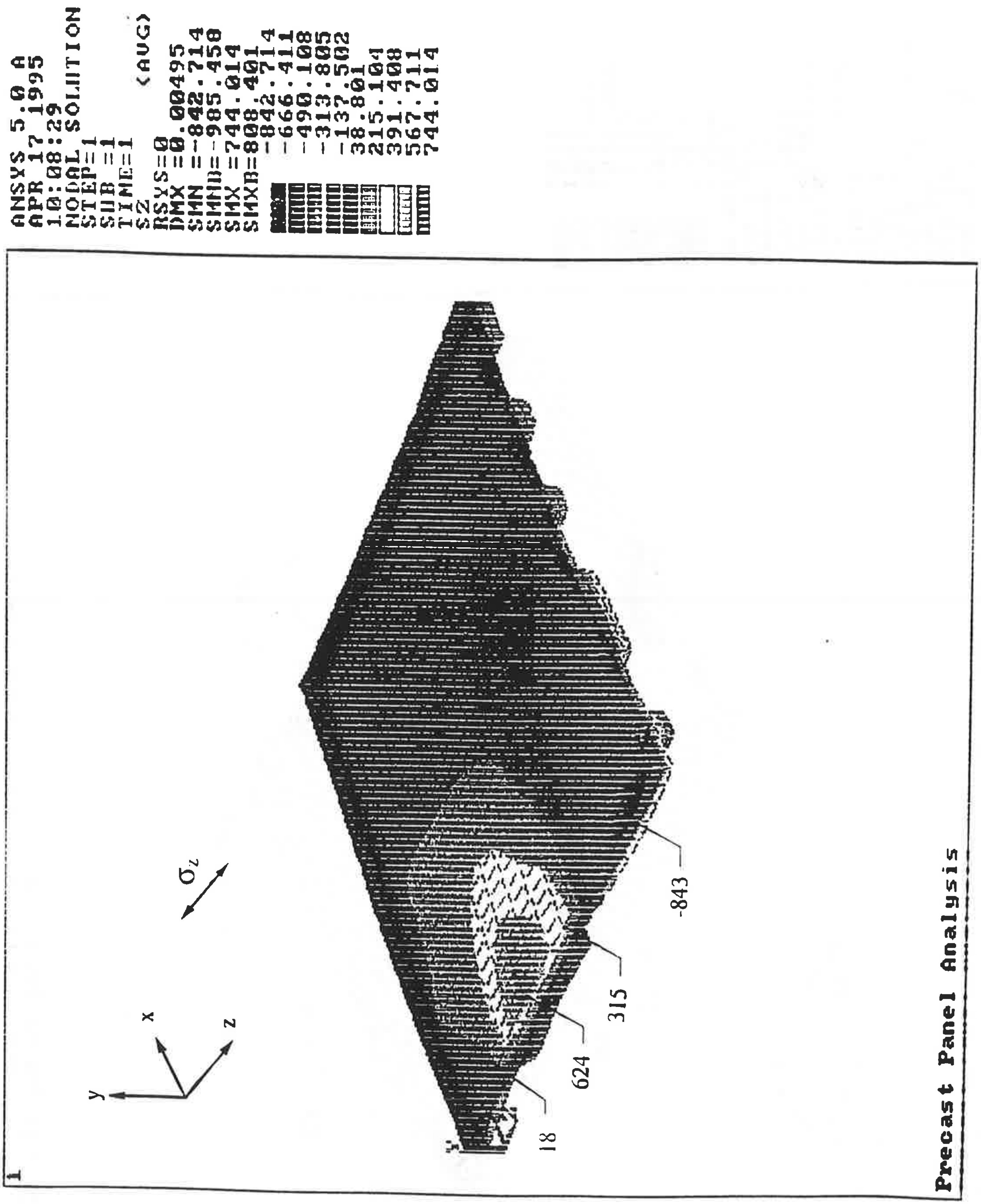


Fig. E.21 Stress in Transverse Direction due to Service Load (top stress)
(all stresses are in psi)

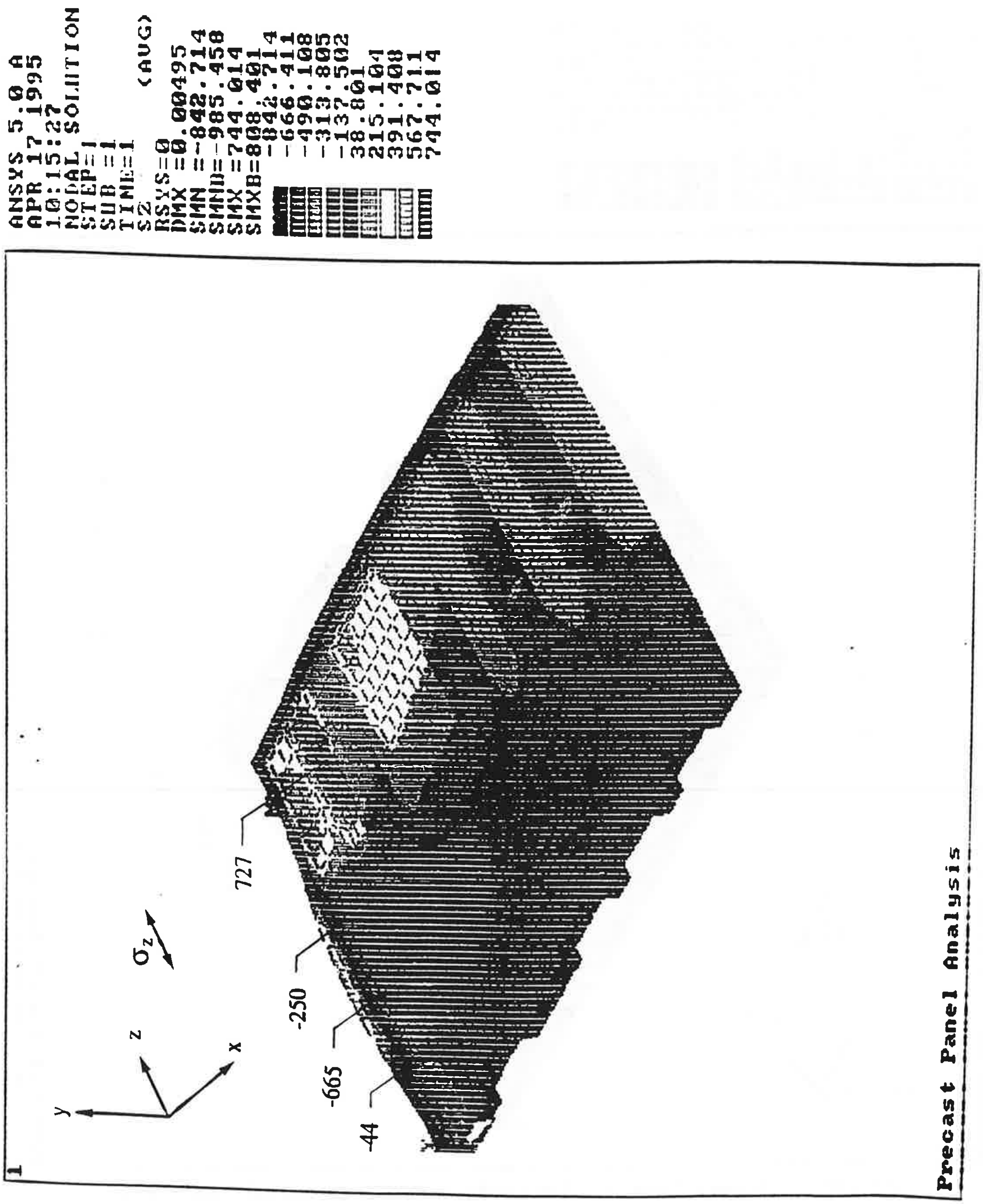


Fig. E.22 Stress in Transverse Direction due to Service Load
(all stresses are in psi)

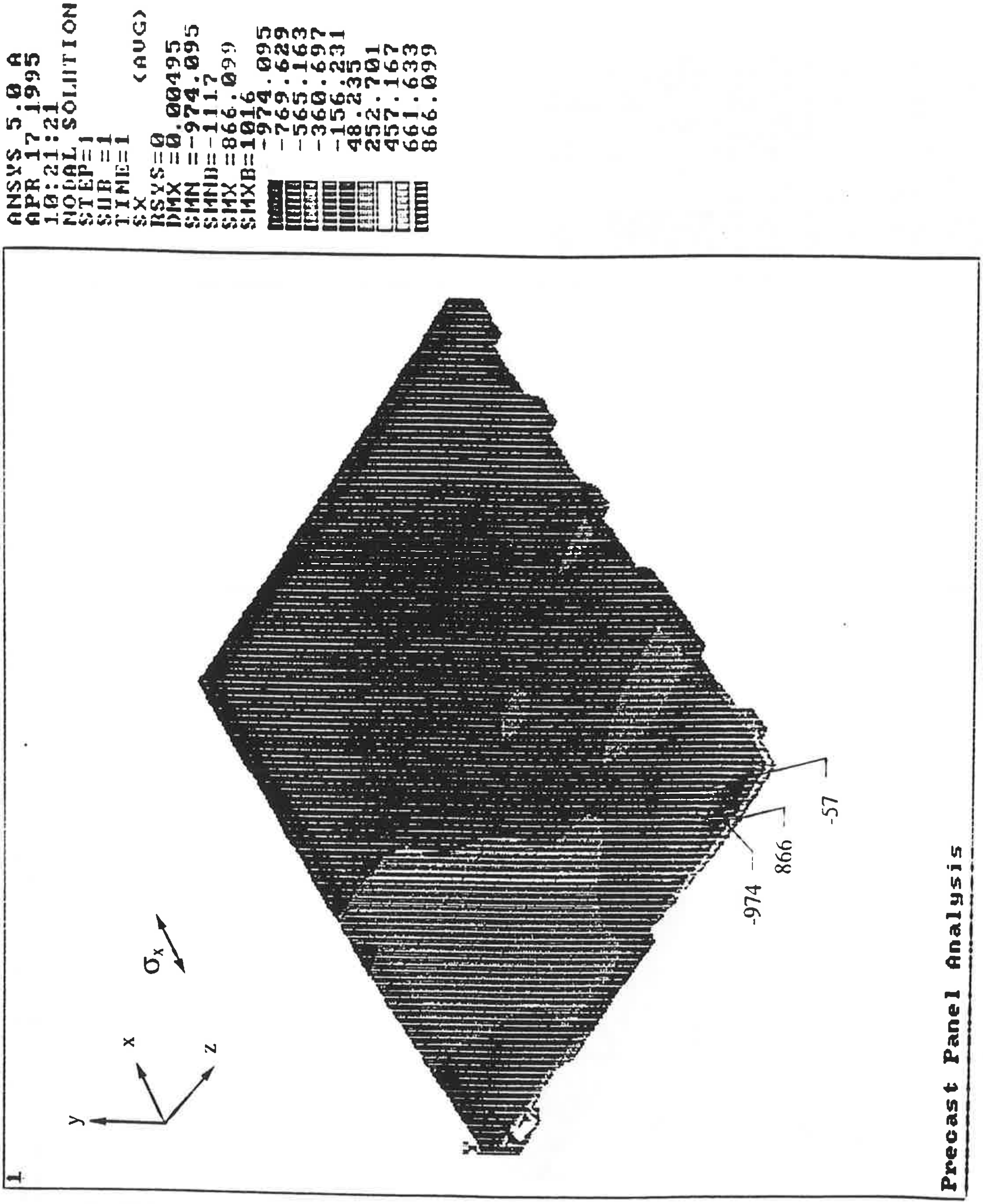


Fig. E.23 Stress in Transverse Direction due to Post-Tensioning and Service Load (all stresses are in psi)

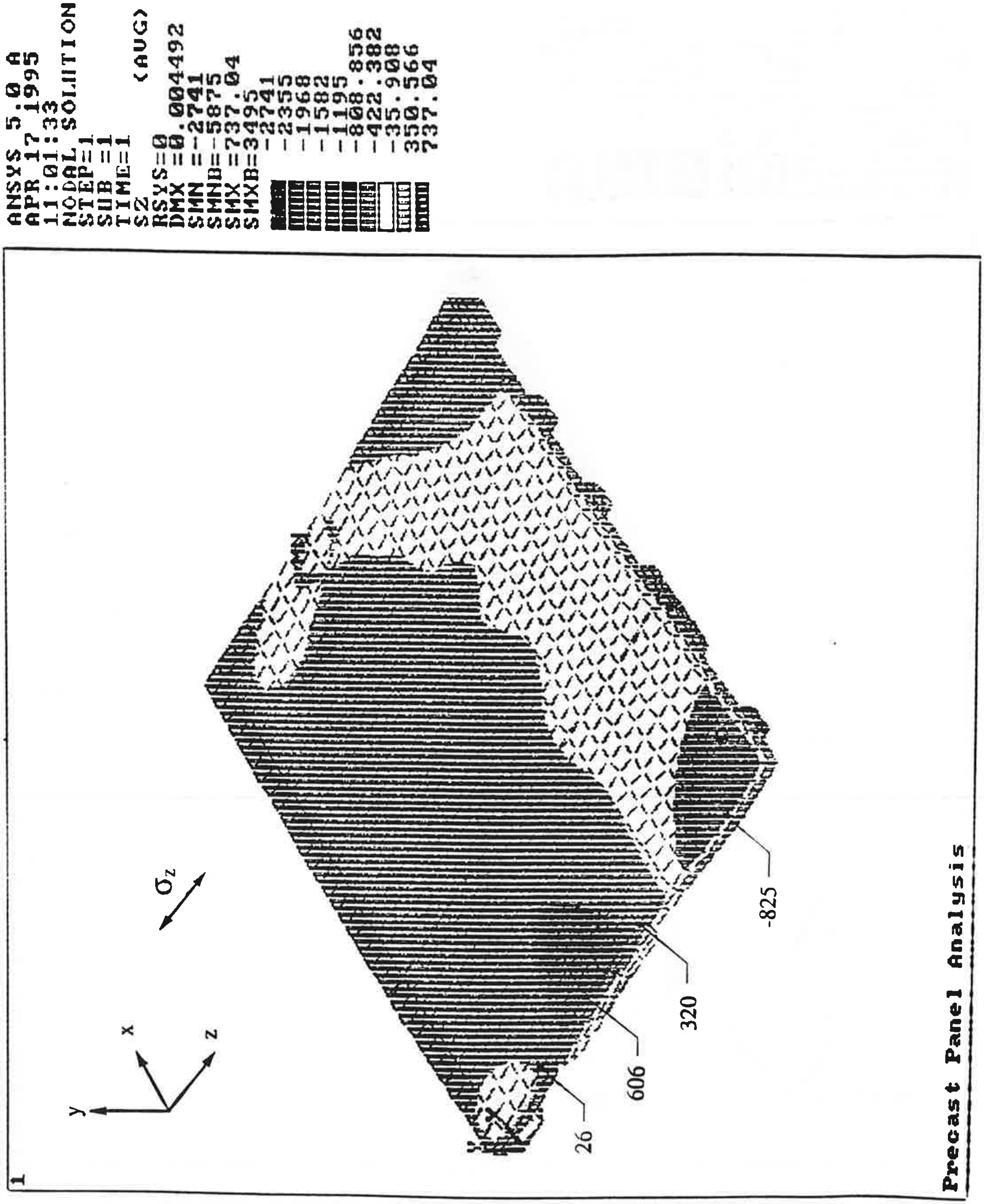


Fig. E.24 Stress in Transverse Direction due to Post-Tensioning and Service Load (all stresses are in psi)

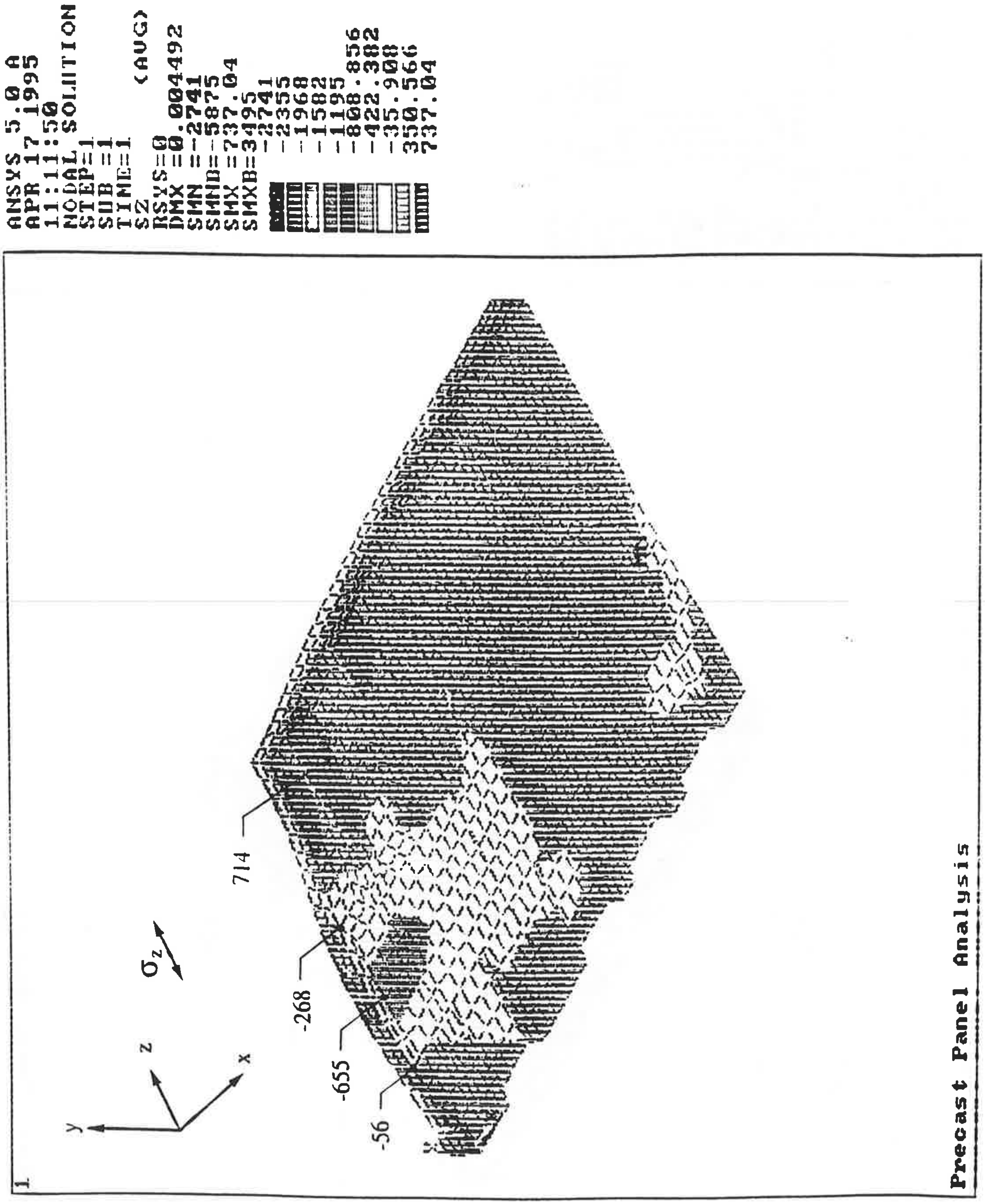


Fig. E.25 Stress in Transverse Direction due to Post-Tensioning and Service Load
(all stresses are in psi)

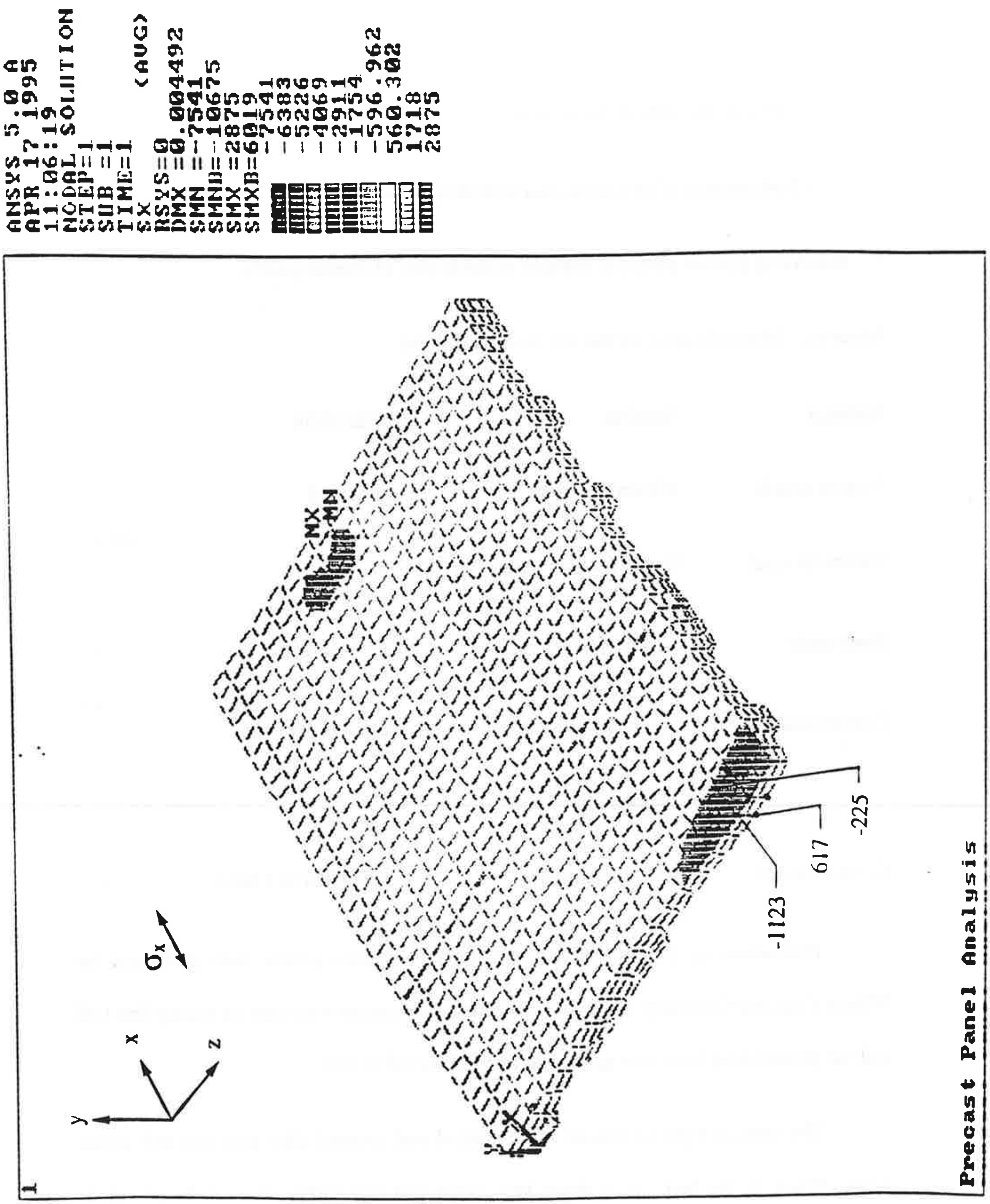


Fig. E.26 Stress in Longitudinal Direction due to Post-Tensioning
(all stresses are in psi)

- Flexural strength of precast panels.

- Performance of post-tensioned transverse shear keys.

* Removing precast panels to confirm removability of precast panels.

Materials: Materials used for the test were as follows:

Material	Supplier	Requirement
Precast panels	Wilson Concrete	As detailed
Structural steel	Drake William's Steel	A36
Steel studs	TSA Manufacturing	$1^1/4$" diameter
Post-tensioning steel	Dywidag Systems International	150K - 1" diameter
Grout (Set 45)	Master Builders	f_c'=2000 psi in 1 hour

Production of Panels: Precast prestressed concrete panels were produced by Wilson Concrete Company, Bellevue, Nebraska. The panels were cast on a long-line bed and the prestressing force was applied to them at the same time.

The bottom layer of strands was installed and stressed after side and end forms were affixed to the bed. Side shear key forms and end forms were made of wood.

Styrofoam was used to form the stemmed shape. Welded wire fabric, post-tensioning ducts and blockouts for post-tensioning anchorages were installed, and then the top layer of strands was installed and stressed.

The locations of post-tensioning ducts were carefully checked and affixed to the top layer of strands. Finally, leveling bolts, lifting bolts, and grouting ducts at each girder location were installed at the designated locations.

7500 psi concrete was cast and steam cured for 14 hours. Prestress was transferred to the panels by torch cutting the strands three days after casting concrete. Both sides of the panels were sandblasted to provide clean and rough surfaces for the transverse grout joint. Figs. E.27 through E.30 show various stages of the production process.

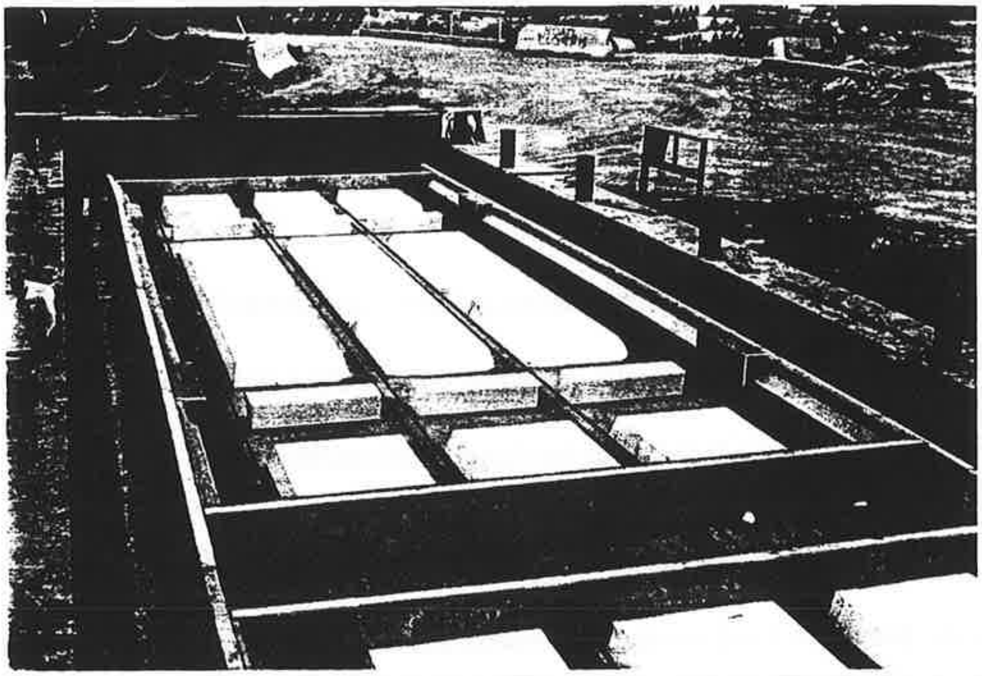


Fig. E.27 Bottom Layer of Strands and Styrofoam

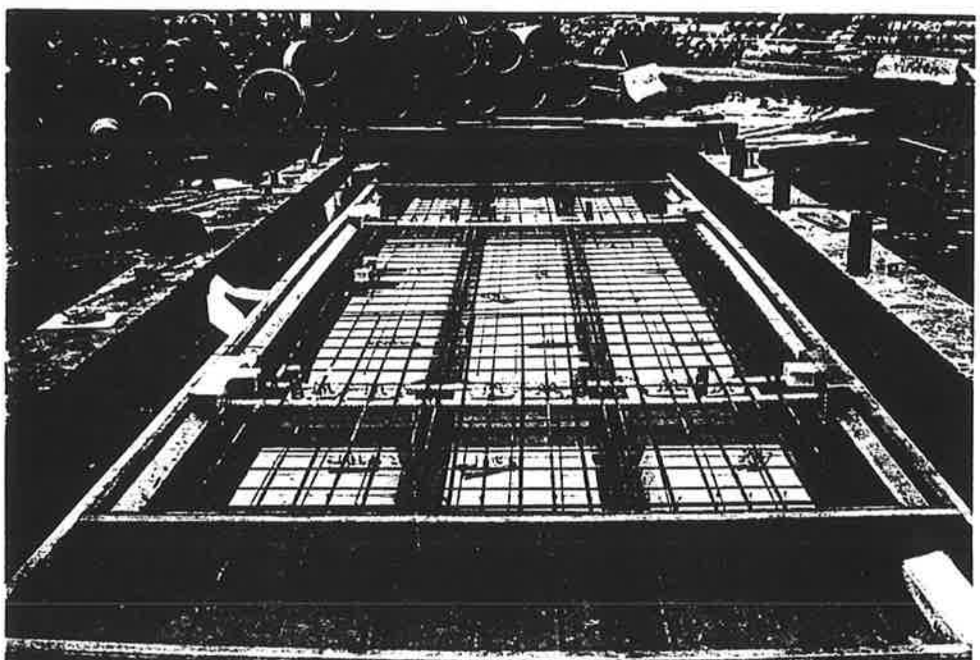


Fig. E.28 Steel Arrangement

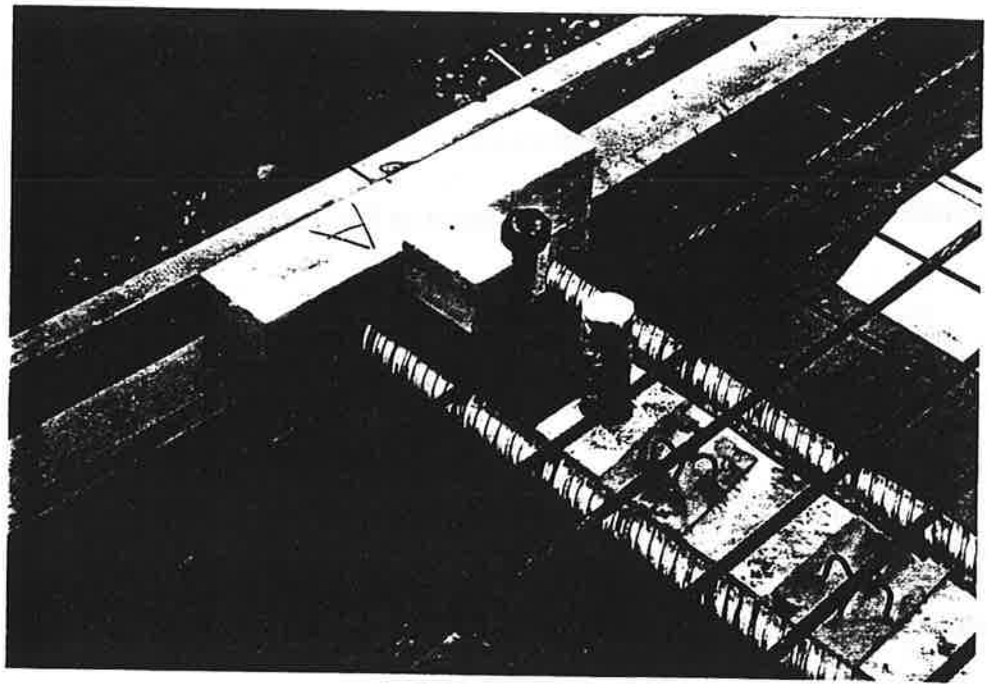


Fig. E.29 Details at a Girder Location (lifting bolt, grout hole, leveling bolt)

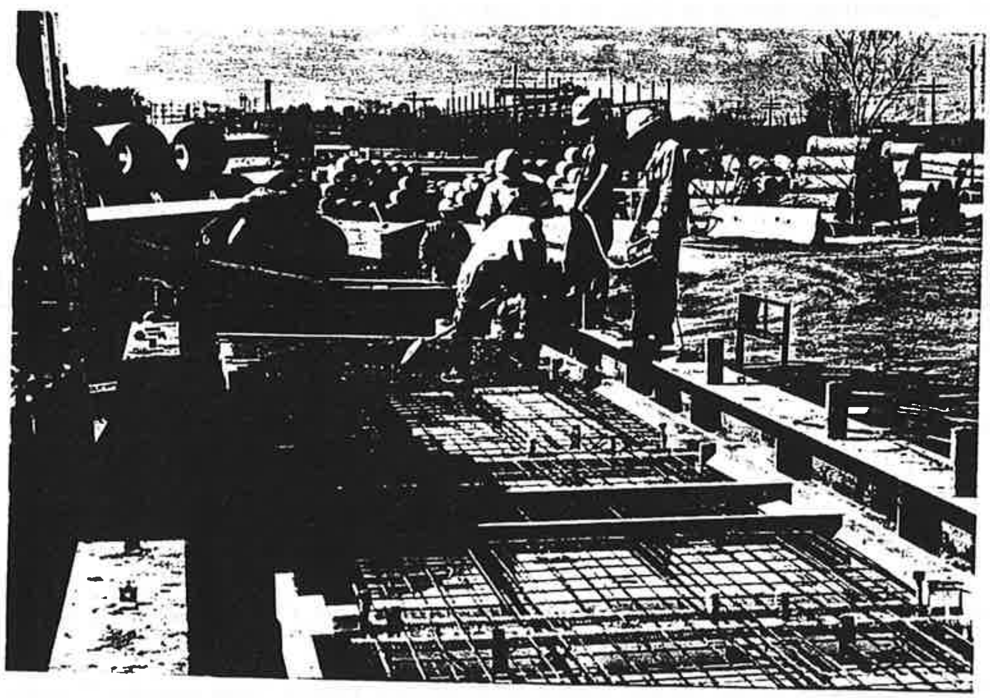


Fig. E.30 Concrete Casting

Transportation of Panels: The panels were transported from Wilson Concrete to the laboratory at the University of Nebraska at Omaha by a trailer truck. An overall view of the trailer truck with precast panels is shown in Fig. E.31.

First stage of construction: Before the precast panels were erected, $1^1/_4$ in. headless studs and threaded studs were welded at their designated locations on the top flange of the steel stringers. Fig. E.32 shows the welding process; the grout stops were then installed on the top flanges of the stringers. The grout stop used was a tubular elastic material (pipe insulation) mounted on the top flange by double sided tape. A release agent was also applied onto the top flange and onto the welded studs to provide for easier deck removal as shown in Figs. E.33 and E.34.

The first two panels were placed on the stringers by an overhead crane. The elevation of the panels was then adjusted by turning leveling bolts which were located at both ends of the panels at girder locations, as shown in Figs. E.35, E.36, and E.37. After post-tensioning bars were installed, post-tensioning ducts were spliced at the transverse joint, as shown in Fig. E.38.

Rapid-set non-shrink grout (Set-45 Hot Weather Mix) with M gravel was then poured into the transverse joint, as shown in Fig. E.39. The Set-45 Hot Weather Mix is a specially blended grout which keeps workability longer than regular Set-45 under high temperatures. This grout's compressive strength at 3 hours was 500 psi (3.44 MPa) and its 24 hour strength was about 2000 psi (13.79 MPa). Based on this performance, it was decided that the product was unsatisfactory for rapid redecking. Regular Set-45, however, showed excellent performance in later stages of construction and should be used for this application.

After the grout obtained a 2000 psi (13.79 MPa) compressive strength, post-tensioning bars were tensioned by hydraulic jack, as shown in Fig. E.40, and concrete strains were recorded using installed strain gages. An 80 kip (17.98 kN) post-tensioning force was applied to the bars. Resulting concrete stresses at the transverse joint were 150 to 200 psi (1.03 to 1.38 MPa). These results agreed with the results from the finite element analysis.

The haunch and blockouts for the headless studs were filled with regular Set-45 without coarse aggregate, as shown in Fig. E.41. For this process, the amount of water was increased by 10 percent for the grout mixture to obtain flowability. The grout was then poured in one grout hole and vented out the other two holes. Even though 10 percent more water than the designated amount was used for the grout mix, the compressive strength of the grout at one hour was more than 3000 psi (20.69 MPa). This performance satisfies the time constraints of a rapid redecking process.

Second stage of construction: To simulate staged construction the third panel was installed after installation of the first two was completed. At the transverse joint between the second and the third panel, post tensioning bars were installed and coupled with those from first construction phase. The remaining second stage construction steps were the same as the first stage of construction. The construction log showing the duration of each task and number of workers, for each construction phases, is shown in Tables E.4 and E.5.

Table E.1 Construction log for first stage (Two panels)

Duration (min.)	Event	Workers
15	Install grout stop and apply release agent	2
10	Erect two precast panels	4
28	Adjust elevation of panels and install nuts for threaded studs	2
12	Install post-tensioning bars (Include anchor plates, nuts, duct splicing)	2
22	Set forms for transverse joint	2
90	Grout transverse joint	3
	(Grout cure and strain-gage installation)	
20	Apply post-tensioning	3
70	Grout haunch and blockouts for headless studs	3
	(Grout cure)	
4.5 hr	Total construction time excluding grout curing time	---

Table E.2 Construction log for second stage (One panel)

Duration (min.)	Event	Workers
10	Install grout stop and apply release agent	2
5	Erect one precast panel	4
15	Adjust elevation of panel and install nuts for threaded studs	2
12	Install post-tensioning bars (Include anchor plates, nuts, couplers, duct splicing)	2
40	Grout transverse joint	3
	(Grout cure and strain-gage installation)	
25	Apply post-tensioning	3
40	Grout haunch and blockouts for headless studs	3
	(Grout cure)	
2.5 hr	Total construction time excluding grout curing time	---

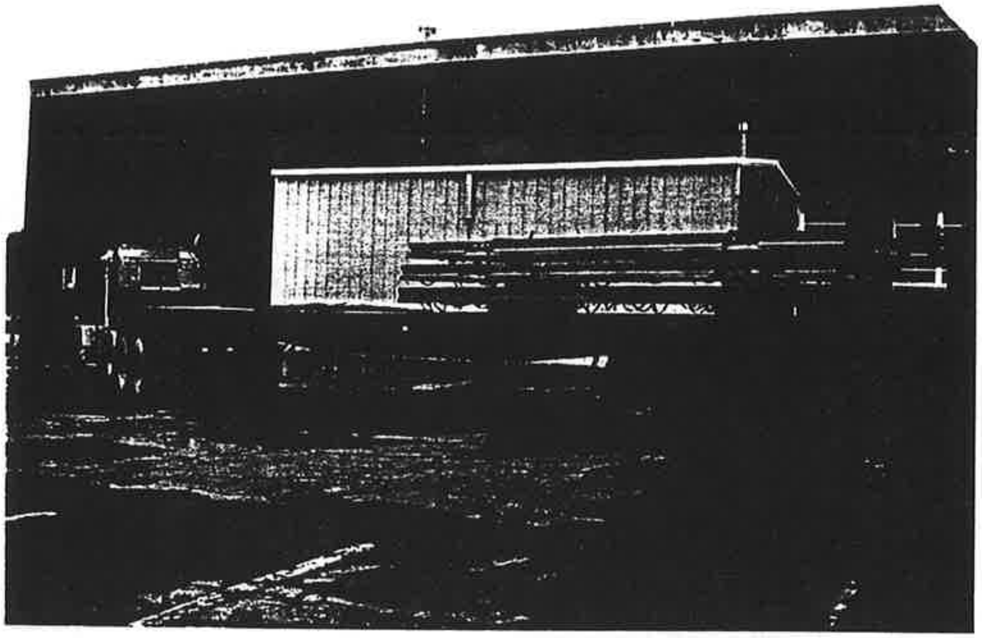


Fig. E.31 Precast Panels on Trailer

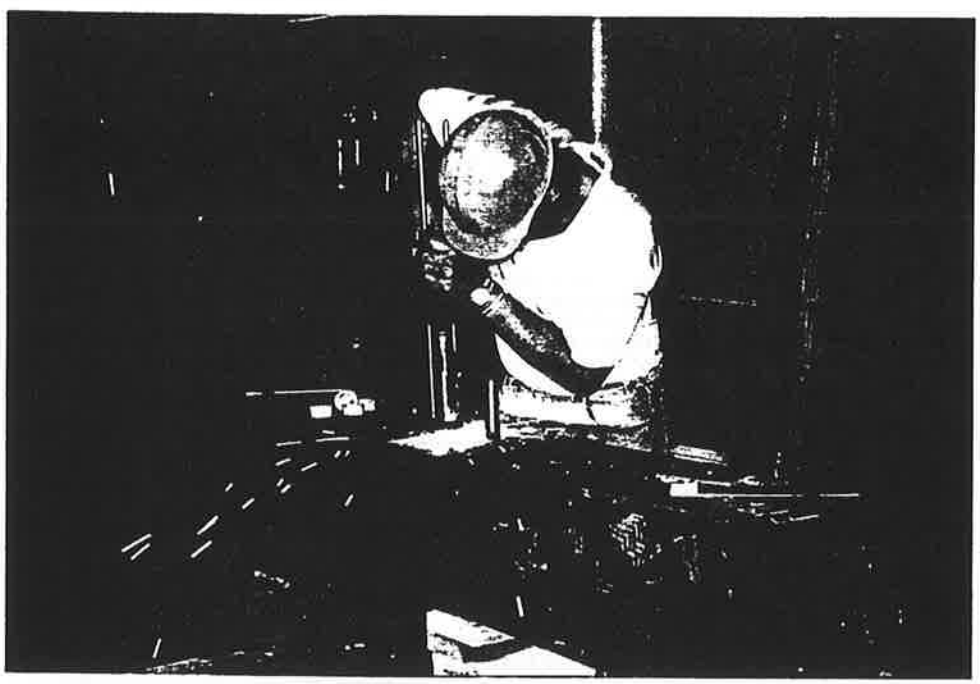


Fig. E.32 Stud Welding

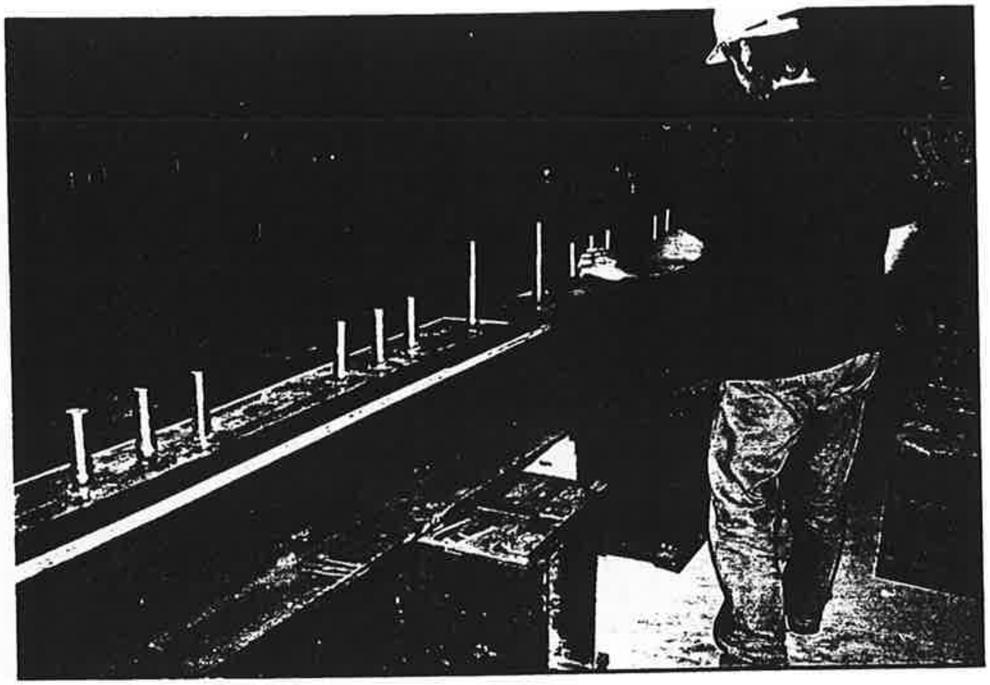


Fig. E.33 Installation of Grout Stop

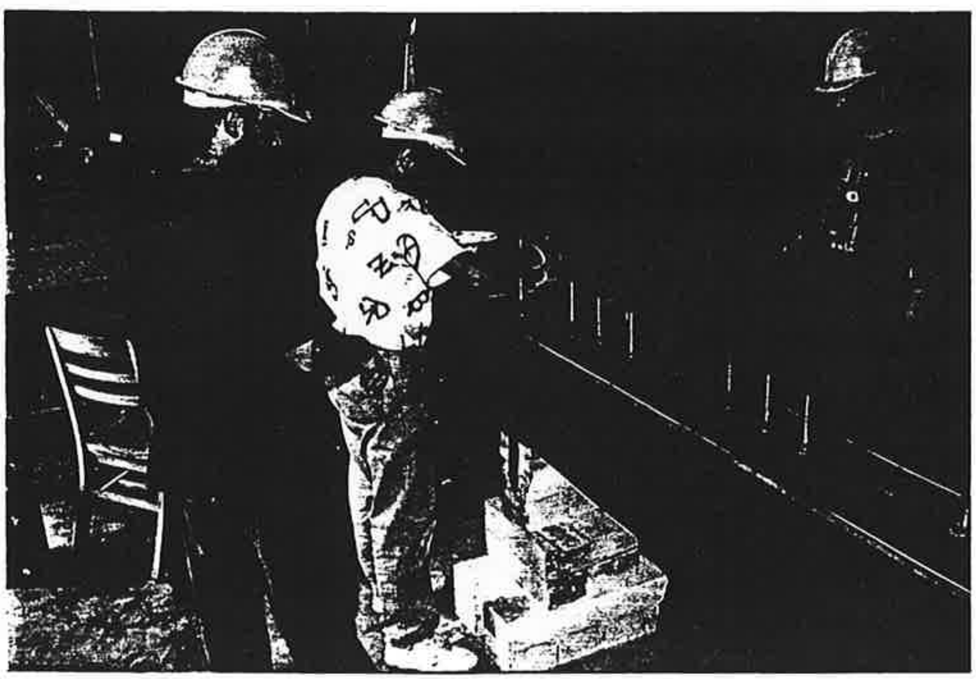


Fig. E.34 Applying Release Agent

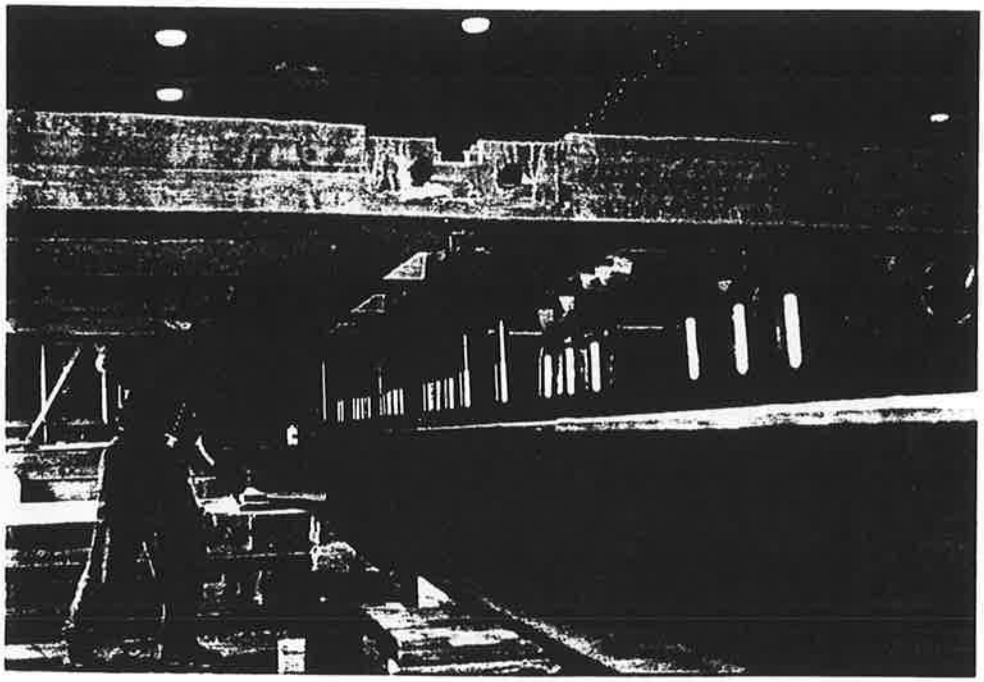


Fig. E.35 Panel Erection

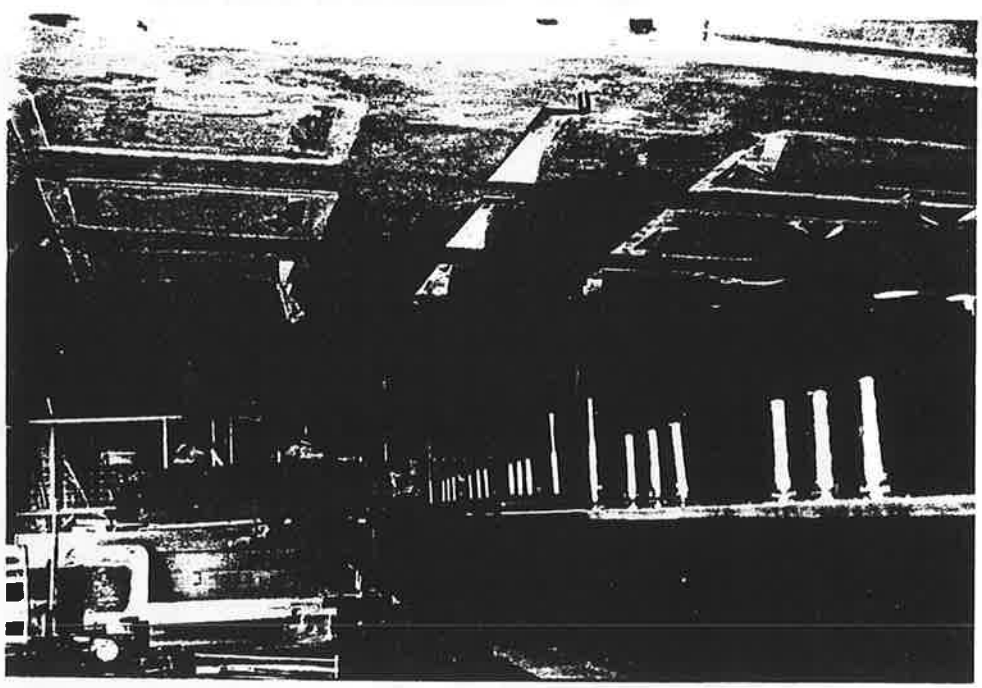


Fig. E.36 Blockouts for Headless Studs

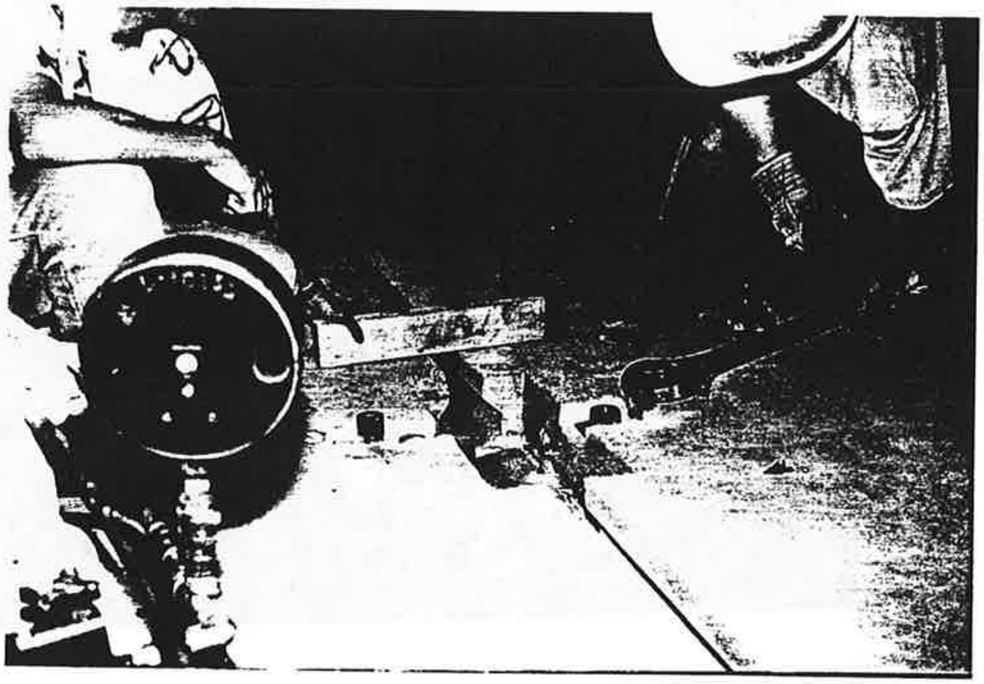


Fig. E.37 Leveling Panels

Fig. E.38 Installation of Post-tensioning Bars

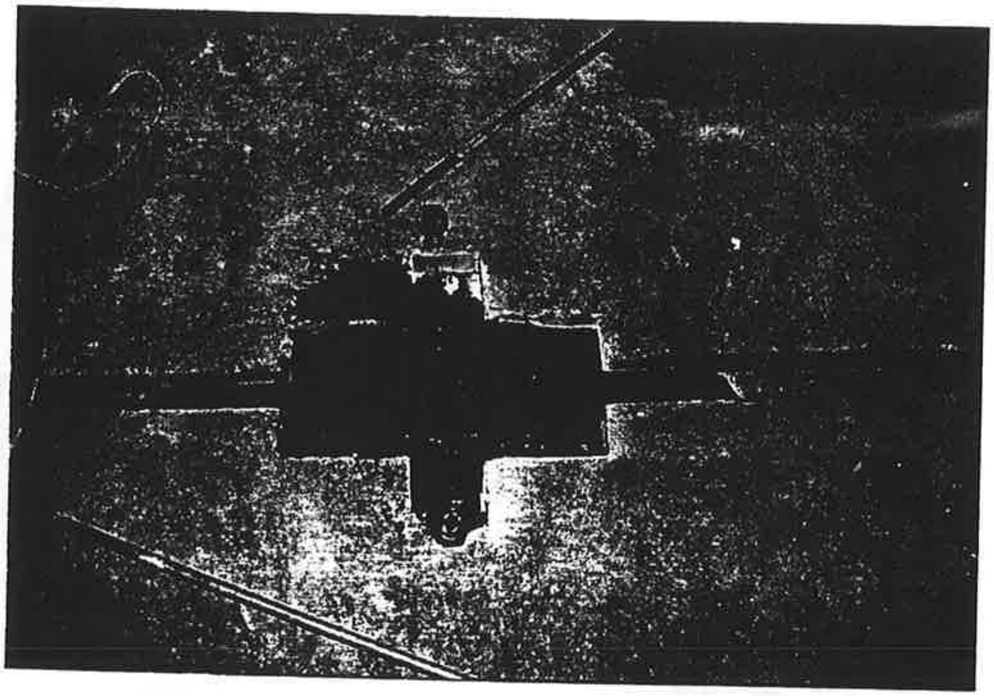


Fig. E.39 Casting Rapid-set Non-shrink Grout (Set 45)

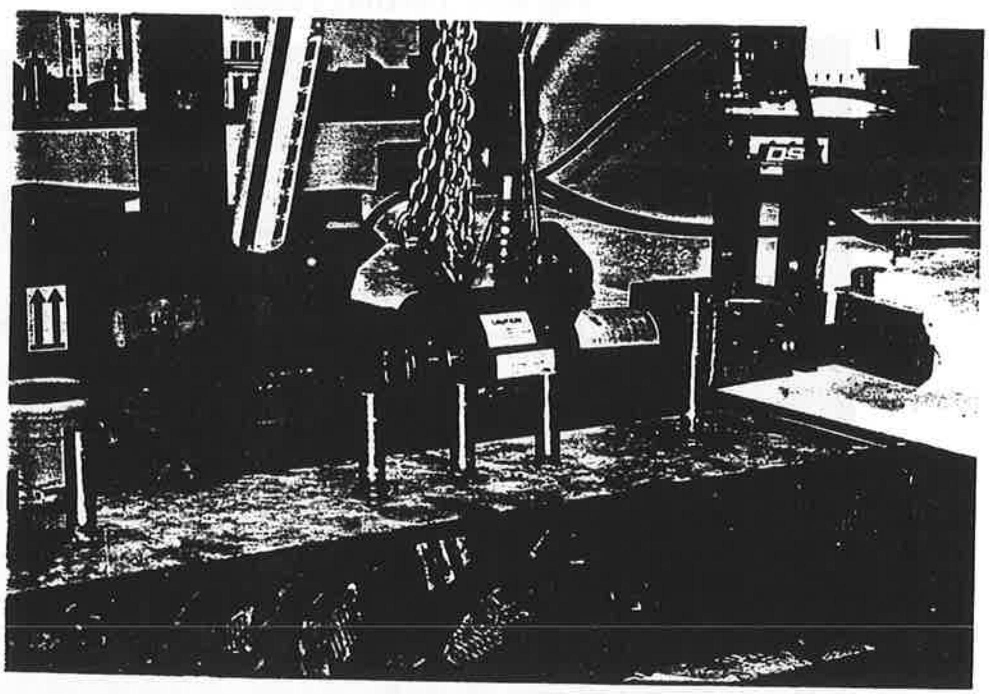


Fig. E.40 Longitudinal Post-tensioning

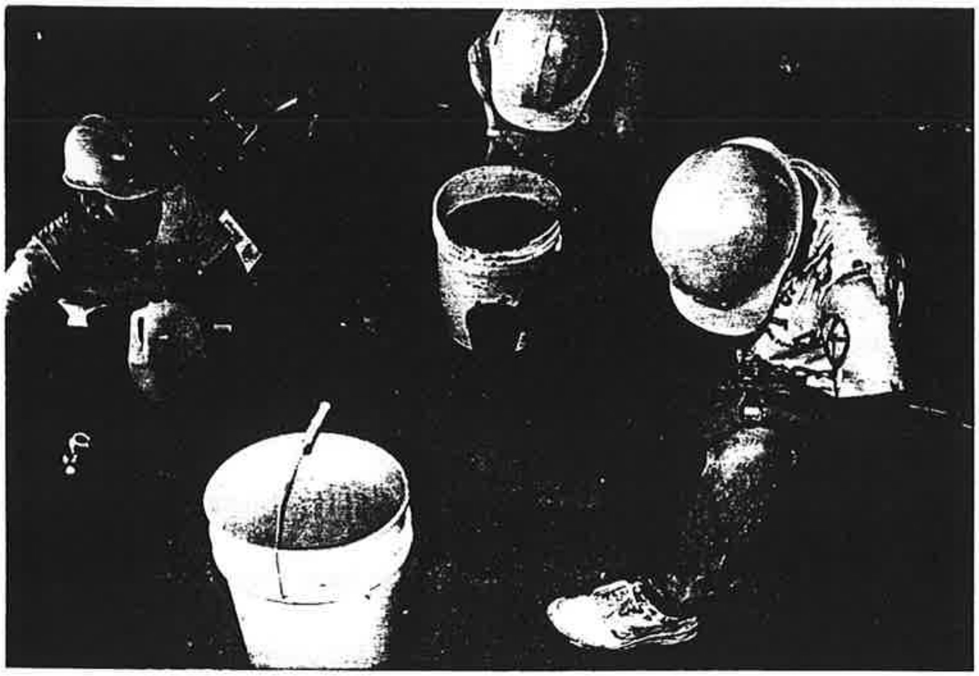


Fig. E.41 Grouting Haunch and Blockouts for Headless Studs

Load Testing:

A simulated axle loading consisting of four concentrated loads in accordance with AASHTO specifications was applied as shown in Fig. E.42. Three loading locations were used. Location 1 was adjacent to a transverse joint, Location 2 was centered between transverse joints, and Location 3 was at the edge of a precast panel. Fig. E.43 shows the loading locations.

Beginning at Location 1, a service load of 25 kips (5.62 kN) per loading point, which simulated the rear wheel of an HS25 load plus impact, was applied monotonically to find a stress distribution over the precast panels due to the concentrated loads. A two million cycle fatigue loading was then conducted at this location. The monotonic service load was again applied at Location 1 to compare the results with those before fatigue loading. A water pool was provided at the transverse joint, as shown in Fig. E.43, to check for water leakage during the fatigue loading.

At Location 3, only the service load was applied to check stress levels in the panels. The purpose of this loading was to simulate truck loading at the joint between an existing deck and newly constructed deck panels simulating construction using daytime opening and nighttime closure. The monotonic ultimate load was applied at Location 2. A series of strain gages was installed on the specimen before loading. The location of each strain gage is shown in Fig. E.44 and E.45. These strain gages were used to measure stresses corresponding to the loads at each location. Displacement gages were also installed to measure the displacement of the precast panels. These gages measured deflections at the tip of the cantilever and mid-span between girders as shown in Fig. E.47.

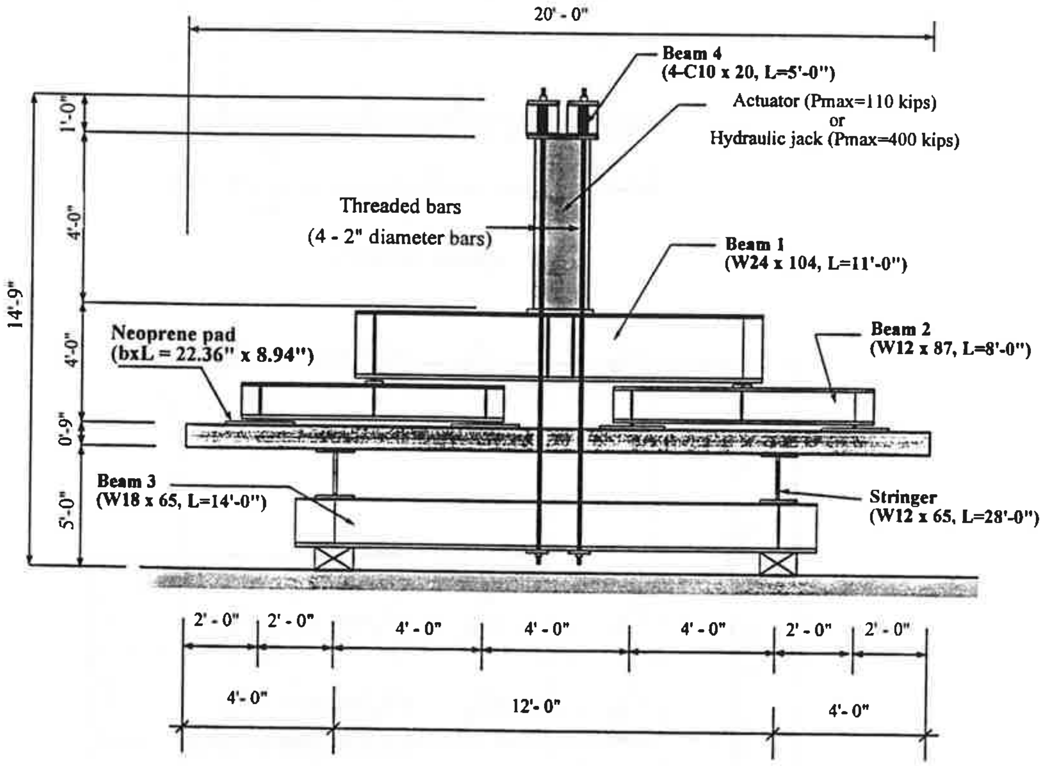


Fig. E.43 Elevation of Test Set Up

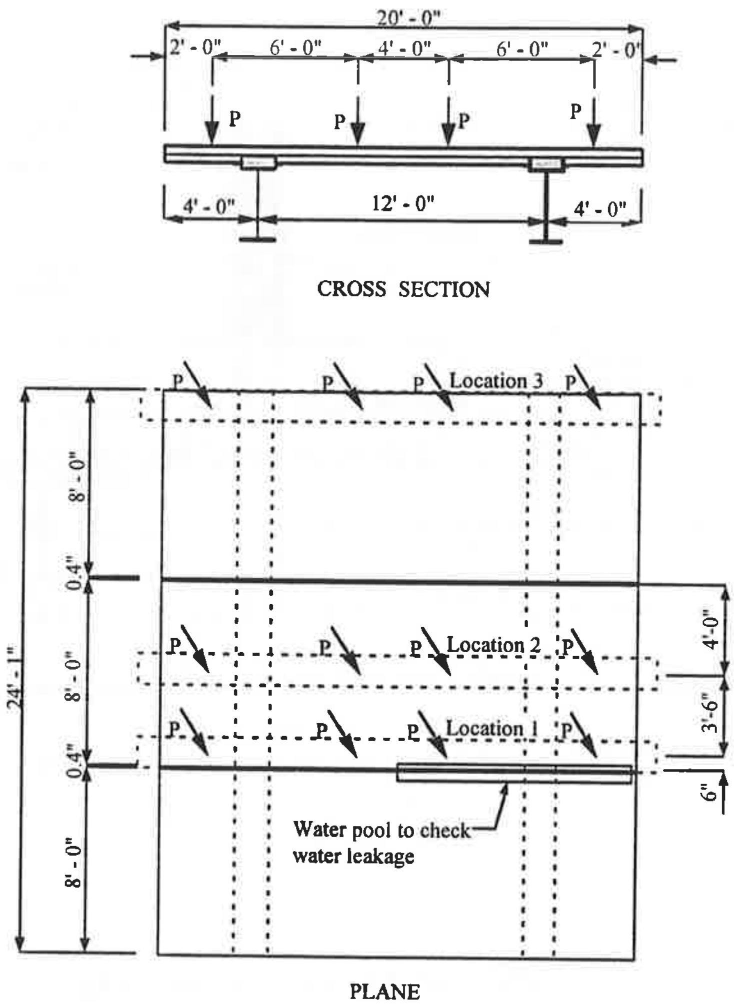
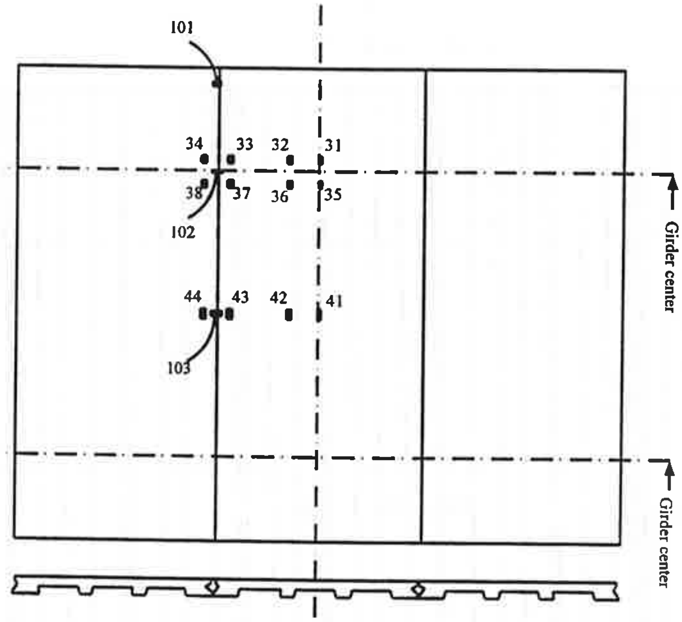
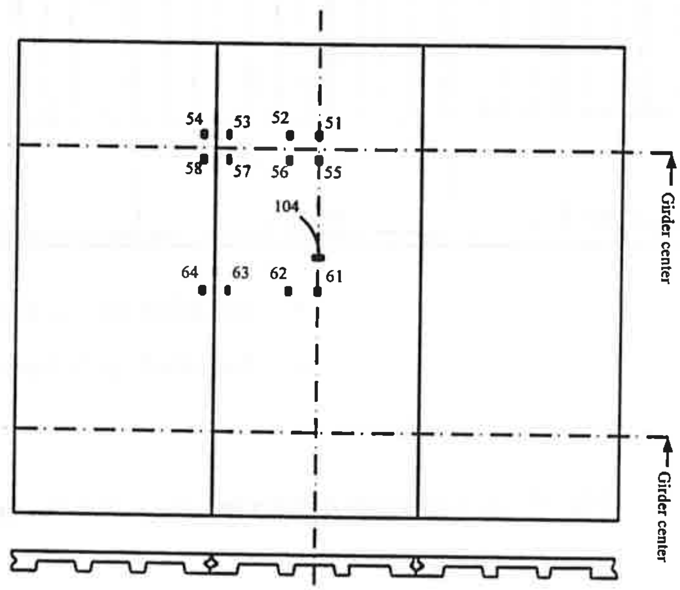


Fig. E.44 Loading locations



Top surface



Bottom surface

Fig. E.45 Location of Strain Gages for Concrete Surface

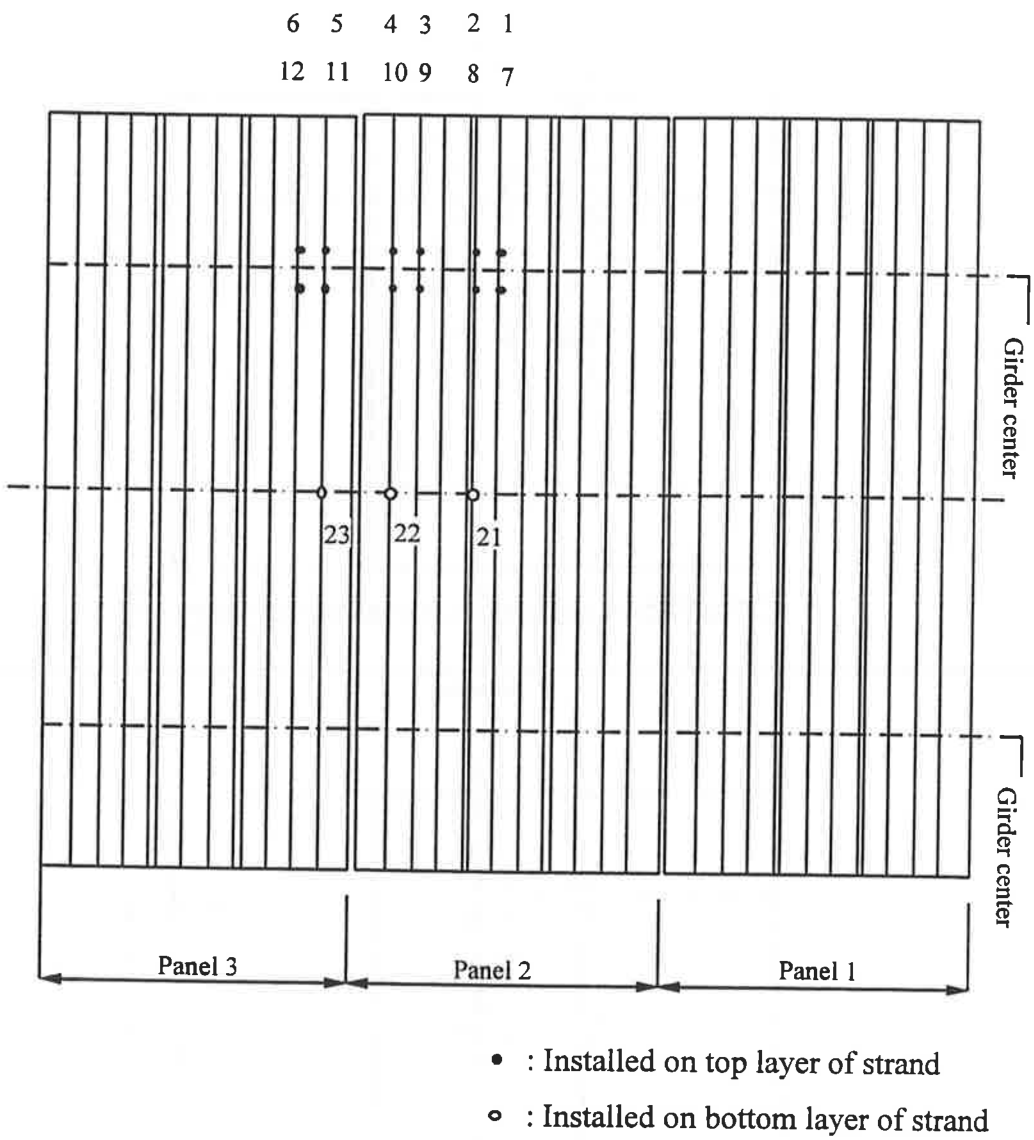


Fig. E.46 Location of Strain Gages for Strands

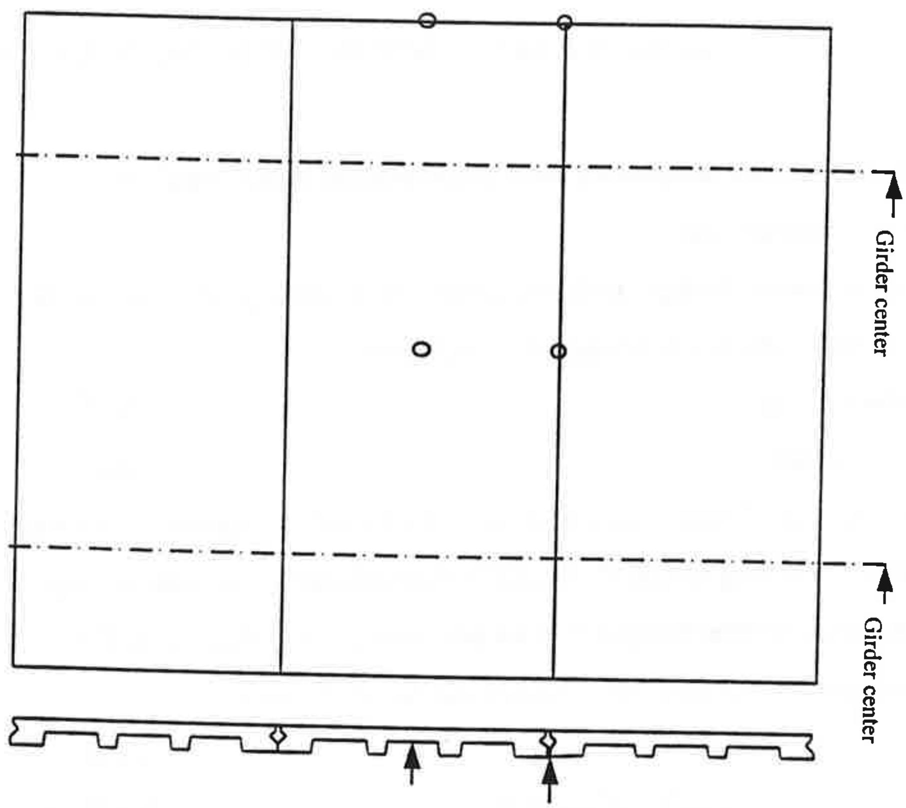


Fig. E.47 Location of Displacement Gages

APPENDIX F

DESIGN CALCULATIONS OF DECK SYSTEMS

9 inch CIP slab reinforced with conventional reinforcement

Design Assumptions

A 44.0 ft-wide bridge deck supported by 4 steel girders spaced at 12 ft and has two overhangs each of 4.0 ft length is considered.

Girder spacing	=	12.0 ft
Overhang length	=	4.0 ft

A 12 in.-wide flange steel girders are considered as supporting elements of the slab.

Design according to LRFD AASHTO Specification, 1st edition, strip design method.

Specified concrete strength at time of opening the bridge for traffic = 4.0 ksi

Conventional reinforcement ASTM-A616, black steel:

Yield strength	=	60 ksi
Modulus of elasticity	=	29,000 ksi
Top reinforcement clear cover	=	2.5 in.
Bottom reinforcement clear cover	=	1.0 in.

A 1/2 in. wearing surface is considered to be an integral part of the 9 in. slab

2 in. concrete future wearing surface is considered

Minimum slab thickness (LRFD Art. 9.7.1.1)

for interior spans	=	7 in. + 0.5 in. sacrificial layer	= 7.5 in.
For overhangs	=	8 in.	
Try slab thickness	=	9 in.	

Loads:

Dead loads (DL):	self weight of 9" CIP slab	= (9/12) (0.150)	= 0.113 kip/ft
	Parapet weight		= 0.660 kip/ft
Wearing surface (WS):			
	self weight of 2" wearing surface	= (2/12) (0.150)	= 0.025 kip/ft

Live loads: It consists of a combination of: (LRFD Art. 3.6.1.2.2)

1. Design truck, equivalent to AASHTO HS-20 truck load and
2. Design lane load of 0.64 KLF.

For cantilevers, a minimum distance of 12 inches from center of wheel to the inside face of parapet should be considered. (LRFD Art. 3.6.1.3).

Multipresense factor: (LRFD Art. 3.6.1.1.2)

Single truck = 1.2

Two trucks = 1.0

Dynamic allowance = 33% (LRFD Art. 3.6.2.1)

Fatigue need not to be investigated for concrete slabs in multi-girder bridges, (Art. 9.5.3 and 5.5.3.1).

Load Combination (LRFD Art. 3.4)

$$U = 1.25(DL) + 1.50(WS) + 1.75(L+I)$$

Design of positive moment section (between girders)

- Effective span: (LRFD Art. 4.6.2.16)

Effective span = Girder spacing - 2 (1/4 girder width)

= 12 ft - 2(0.25 x 1 ft) = 11.5 ft

- Bending moment: (LRFD Art. 3.4)

DL: Due to self weight of slab, $M_s = 0.113\ (11.5^2/10)$ = 1.494 ft-kips

WS: Due to wearing surface, $M_{ws} = 0.025\ (11.5^2/10)$ = 0.331 ft-kips

LL: From Table A4.1-1 in AASHTO LRFD Specification, for S = 12.0 ft

Max. positive bending moment with impact M_{L+I} = 8.010 ft-kips/ft

Therefore, maximum positive service bending moment

= 1.494+0.331+8.01 = 9.835 ft-kips/ft

Maximum positive factored bending moment =

M_u = 1.25x1.494 + 1.5x0.331 + 1.75x8.01 = 16.381 ft-kips/ft

- Design of section:

Assume # 5 bar, and 1 in. clear cover, therefore:

$d \quad = 9 - 0.5x0.625 - 1.0 - 0.5 \qquad = \quad 7.19$ in.

$R_n \quad = (M_u / \phi bd^2) \qquad = (16.382x12)/(0.9x12x7.19^2) \qquad = \quad 0.352$ kip/in^2

$m \quad = (f_y / 0.85f_c') \qquad = (60)/(0.85x4.0) \qquad = \quad 17.65$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-2\frac{mR_n}{f_y}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2x17.65x0.352}{60}}\right) = 6.2 \times 10^{-3}$$

$A_s \quad = \rho\,(bd) \qquad = (6.2 \times 10^{-3})(12x7.19) \qquad = \quad 0.53$ in.^{2}

Use #5 bars @ 7 in., $A_s \quad = \quad 0.31\ (12/7) \qquad = \quad 0.53$ in.^{2}

$a \quad = (A_s\, f_y) / (0.85\, b\, f_c') \quad = (0.53x60) / (0.85x12x4.0) \qquad = \quad 0.78$ in.

$\varphi M_n \quad = 0.9\, (A_s\, f_y)\, (d - a/2) \quad = 0.9(0.53x60)(7.19-0.5x0.78)/12 \qquad = \quad 16.300$ ft-kips

$\qquad \cong \quad M_u$ OK

- Check maximum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

$d_e \quad = \quad 7.19$in.

$c \quad = \quad (a / \beta) \quad = \quad (0.78/0.85) \quad = \quad 0.917$

$c / d_e \quad = \quad (0.917/7.19) \quad = \quad 0.127 \quad < \quad 0.42$ OK

- Check minimum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

$\rho_{act} \quad = \quad 0.53 / (12 \times 7.19) \quad = \quad 6.14 \times 10^{-3}$

$\rho_{min.} \quad = \quad 0.03\, (f_c' / f_y) \quad = (0.03x4.0)/(60) \quad = \quad 2.00 \times 10^{-3} \quad < \rho_{act}$ OK

- Check of crack control by distribution of reinforcement: (LRFD Art. 5.7.3.4)

$d_c \quad = (1 + 0.5 \times 0.625) \quad = \quad 1.3125$ in.

$A_c \quad = (2 \times 1.3125 \times 7) \quad = \quad 18.37$ in.^{2}

$Z \quad = \quad 170$ kip/in

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = \frac{170}{\sqrt[3]{18.37x1.3125}} = 58.8 \text{ ksi} > (0.6\, f_y = 36 \text{ ksi})$$

Thus $f_{sa\ all.} \quad = \quad 36$ ksi

Actual stress in reinforcement:

$E_c \quad = \quad 33{,}000\, (w_c)^{1.5} \sqrt{f_c'}$ (LREFD Eq. 5.4.2.4-1)

$\quad = 33{,}000\, (0.150)^{1.5} \sqrt{4.0} \quad = \quad 3835$ ksi

$n \quad = \quad E_s / E_c \quad = 29{,}000/3835 \quad = \quad 7.56$

$\rho_{act} \quad = \quad 6.14 \times 10^{-3}$

$k = \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.308$

$j = (1 - k/3) = 0.897$

$f_{sa\ act.} = M_{service} / (jdA_s)$

$= (9.835x12) / (0.897x7.19x0.53) = 34.5$ ksi $< f_{sa\ all.}$ OK

- Check fatigue limit state: (LRFD Art. 5.5.3)

Fatigue need not to be investigatedfor concrete deck slabs in multi-girder applications.

Design of section of negative moment section over interior girder lines:

- Critical section for negative moment over the girders is at a distance of

(steel flange width / 4) = 3 in. (From the center line of the girder).

Effective span = 12.0 - 2(3/12) = 11.5 ft

- Bending moment: (LRFD Art. 3.4)

DL: Due to self weight of slab, $M_s = 0.113\ (11.5^2/10)$ = 1.494 ft-kips

WS: Due to wearing surface, $M_{ws} = 0.025\ (11.5^2/10)$ = 0.331 ft-kips

LL: From Table A4.1-1 in AASHTO LRFD Specification, for S = 12.0 ft

Max. negative bending moment with impact at 3 in. from girder ceneter line,

M_{L+I} = 9.400 ft-kips/ft

Therefore, maximum negative service bending moment

= 1.494+0.331+9.400 = 11.225 ft-kips/ft

Maximum positive factored bending moment =

M_u = 1.25x1.494 + 1.5x0.331 + 1.75x9.400 = 18.814 ft-kips/ft

- Design of section:

Assume # 5 bar, and 2.5 in. clear cover, therefore:

$d = 9 - 0.5x0.625 - 2.5 = 6.19$ in.

$R_n = (M_u / \phi bd^2) = (18.8x12)/(0.9x12x6.19^2) = 0.545$ kip/in^2

$m = (f_y / 0.85f'_c) = (60)/(0.85x4.0) = 17.65$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-2\frac{mR_n}{f_y}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2x17.65x0.545}{60}}\right) = 9.96 \times 10^{-3}$$

$A_s = \rho\ (bd) = (9.96 \times 10^{-3})(12x6.19) = 0.74$ in.^{2}

Use #5 bars @ 5 in., A_s = 0.31 (12/5) = 0.744 in.^{2}

$a = (A_s f_y) / (0.85\, b\, f_c') = (0.744x60) / (0.85x12x4.0) = 1.09$ in.

$\varphi M_n = 0.9\, (A_s f_y)\, (d - a/2) = 0.9(0.744x60)(6.19-0.5x1.09)/12 = 18.899$ ft-kips $> M_u$ OK

- Check maximum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

$d_e = 6.19$in.

$c = (a / \beta) = (1.09/0.85) = 1.28$

$c / d_e = (1.28/6.19) = 0.21 < 0.42$ OK

- Check minimum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

$\rho_{act} = 0.744 / (12 \times 6.19) = 10.016 \times 10^{-3}$

$\rho_{min.} = 0.03\, (f_c' / f_y) = (0.03x4.0)/(60) = 2.00 \times 10^{-3} < \rho_{act}$ OK

- Check of crack control by distribution of reinforcement: (LRFD Art. 5.7.3.4)

$d_c = (2.5 + 0.5 \times 0.625) = 2.813$ in.

$A_c = (2 \times 2.813 \times 5) = 28.13$ in.^{2}

$Z = 170$ kip/in

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = \frac{170}{\sqrt[3]{28.13x2.813}} = 39.6 \text{ ksi} > (0.6\, f_y = 36 \text{ ksi})$$

Thus $f_{sa\,all.} = 36$ ksi

Actual stress in reinforcement:

$E_c = 33{,}000\, (w_c)^{1.5} \sqrt{f_c'}$ (LRFD Eq. 5.4.2.4-1)

$= 33{,}000\, (0.150)^{1.5} \sqrt{4.0} = 3835$ ksi

$n = E_s / E_c = 29{,}000/3835 = 7.56$

$\rho_{act} = 10.016 \times 10^{-3}$

$k = \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.396$

$j = (1 - k/3) = 0.868$

$f_{sa\,act.} = M_{service} / (jdA_s)$

$= (11.225x12) / (0.868x6.19x0.744) = 33.7$ ksi $< f_{sa\,all.}$ OK

- Check fatigue limit state: (LRFD Art. 5.5.3)

Fatigue need not to be investigatedfor concrete deck slabs in multi-girder applications.

Design of section of negative moment over the exterior girder line (design of the overhang):

- Critical section for negative moment over the girders is at a distance of :

(steel flange width / 4) = 3 in. (From the center line of the girder).

Therefore, cantilever span is = 4' - (3/12) = 3.75 ft

Art. 13.4.1 states that three design cases need to be checked when designing the overhang regions.

- Case # 1: check overhang for horizontal vehicular collision load: (LRFD Art. 13.4.1)

The overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the (collision+dead loads) moment.

Bending moments:

From design of barrier, (Load and Resistance Factor Design for Highway Bridges, page 11-29), collision moment (moment of resistance of the barrier) at the barrier base, M_c = 23.96 ft-kips

Factored bending moment, M_u, at face of barrier due collision and dead loads (see Fig. F-1 for dimensions)

$$= 23.96 + 1.25[\ (0.5x0.113x1.7^2) + 0.66(1.7-0.625)\ ] \qquad = 25.051 \text{ ft-kips}$$

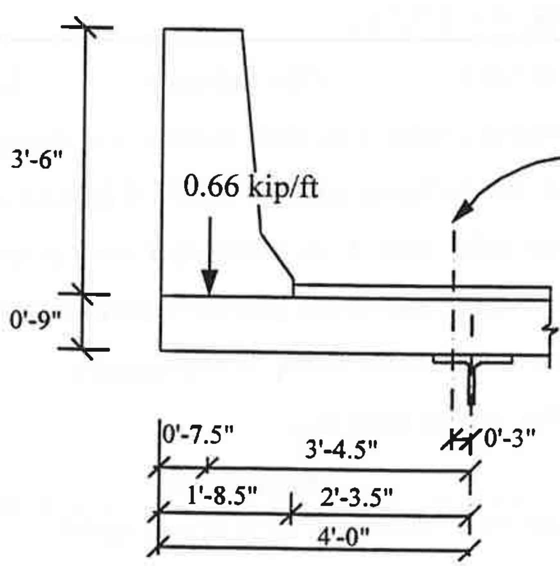


Fig. F-1. Critical Section of the Overhang

Axial tensile force:

Collision horizontal force at top of barrier, R_w = 154.24 kips

This force is distributed over a width of L_c at the top fiber of the barrier = 164.74 in.

Height of the barrier, H = 42 in.

Assume that this force is distributed at an angle of 45° from the top fiber of the barrier to its base, thus:

Collision force at deck slab level, F $= R_w/(L_c+2H)$

$= 154.24 / (164.74+2x42)$ $= 0.62$ kip/in. $= 7.44$ kip/ft

Design of the section at the inner face of barrier:

Try #6 @ 5 in., $A_S = 0.44 \times 12 / 5$ $= 1.056$ in.^{2}

For # 6 bar and 2.5 in. clear cover.

d	= 9 - 0.5x0.75 - 2.5	=	6.125 in.
b		=	12 in.
T	= 1.056 x 60	=	63.36 kips
F		=	7.44 kip/ft
C	= T - F	=	55.92 kips
a	$= C / (0.85bf_c')$	=	1.37 in.

ϕ = 1.0 (for extreme event, Art. 1.3.2.1)

$\phi M_n = \phi[\ T(d-a/2) - F(0.5d-0.5a)\ ] =$ 26.300 ft-kips/ft $> M_u$ OK

For critical section over the exterior girder, 3 in. from centerline of the girder:

At the inner face of the barrier, the collision moment (23.96 ft-kips) is distributed over a lenegth L_c, while the collision axial foce F is distributed over a length of (L_c+2H). Assume that the moment at the face of the barrier and the axial force are distributed at an angle of 30° form the inner face of the barrier to the critical section.

Collision bending moment at the critical section is:

$$= \frac{M_c L_c}{L_c + 2(2.29x12 - 3)\tan 30} = \frac{23.96x164.74}{164.74 + 2(2.29x12 - 3)\tan 30} = 20.5 \text{ ft-kips}$$

Factored bending moment at critical section due to collision force and dead loads:

$M_u \quad = 20.5 + 1.25[\,0.5x0.113(2.29\text{-}0.25)^2 + 0.66(2.29\text{-}0.25)]$

$\qquad + 1.5\,[0.5x0.025(2.29\text{-}0.25)^2] = \qquad 22.555$ ft-kips

Collision axial force at critcal section:

$$F \quad = \frac{R^w}{L_c + 2H_c + 2(2.29 - 0.25)\tan 30}$$

$$= \frac{154.24}{164.74 + 2x42 + 2(2.29 - 0.25)\tan 30} \quad = \quad 0.614 \text{ kip/in.} \quad = 7.4 \text{ kip/ft}$$

Try #6 @ 5 in., $A_S = 0.44 \text{ x } 12 / 5 \quad = \quad 1.056 \text{ in.}^2$

$T \quad = 1.056 \text{ x } 60 \quad = \quad 63.36$ kips

$C \quad = T - F \quad = \quad 55.96$ kips

$a \quad = C / (0.85bf_c') \quad = \quad 1.37$ in.

$\phi \quad = 1.0$ (for extreme event)

$\phi M_n \quad = \phi[\,T(d\text{-}a/2) - F(0.5d\text{-}0.5a)\,] = \quad 42.200$ ft-kips/ft $\quad > M_u \quad$ OK

- Case # 2: check overhang for vertical collision force (LRFD Art. 13.4.1)

For concrete parapets, the case of vertical collision never controls.

- Case # 3: check overhang due to dead and live load: (LRFD Art. 13.4.1)

DL: Due to self weight of slab, $M_s = 0.1125\,(3.75^2/2) \quad = \quad 0.790$ ft-kips

Due to barrier load, $M_b \quad = 0.660\,(\,3.375\text{-}\,3/12\,) \quad = \quad 2.063$ ft-kips

WS: Due to wearing surface, $M_{ws} = 0.025\,(2.29\text{-}3/12)^2\;/\,2 \quad = \quad 0.052$ ft-kips

LL: For maximum negative moment, the truck wheel should be at 12 in. from the face of the barrier, as shown in Fig. F-2.

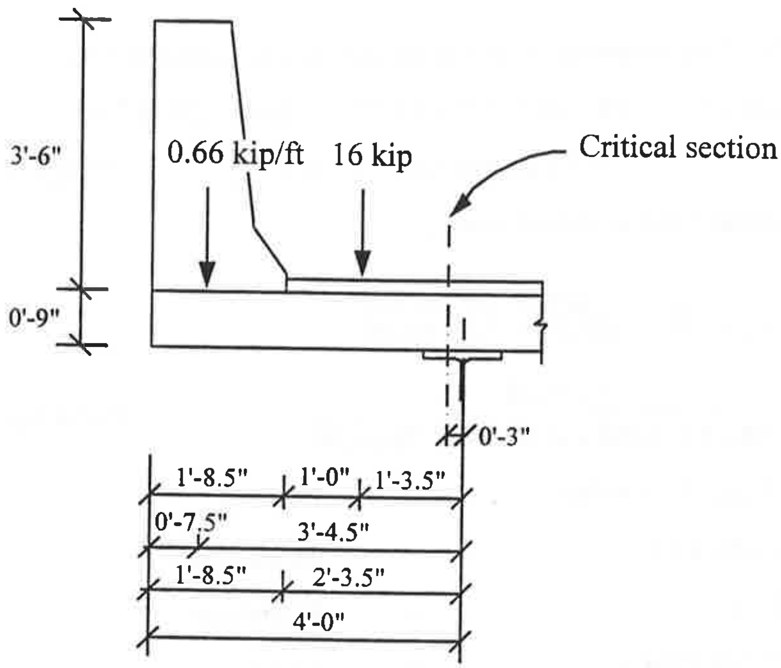


Fig. F-2. Overhang Proportions

Wheel load is distributed over a width of (45.0 + 10 X) (LRFD Art. 4.6.2.1.3)

= 45 + 10 (1.29-3/12) = 55.4 in. = 4.6 ft

Dynamic allowance = 33%, Multi presence factor = 1.2

Max. negative bending moment. M_{L+I} = (16/4.6)(1.292 - 3/12)(1+0.33)(1.2) = 5.79 ft-kips

Therefore, maximum negative service bending moment

= 0.790 + 2.063 + 0.052 + 5.79 = 8.695 ft-kips

Maximum negative factored bending moment

= 1.25(0.790 + 2.063) + 1.5(0.052) + 1.75(5.79) = 13.777 ft-kips

Try # 6 @ 5in., $A_s = 1.056\ in^2$

a = $(A_s f_y) / (0.85\ b\ f'_c)$ = (1.056x60) / (0.85x12x4.0) = 1.553 in.

φM_n = 0.9 $(A_s f_y)$ (d- a/2)

= (0.9)(1.056x60)(6.125-0.5x1.553) /12 = 25.400 ft-kips

> M_u OK

- Check maximum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

d_e = 6.125 in.

c = (a / β) = 1.553/0.85 = 1.827

c / d_e = 1.827/6.125 = 0.30

< 0.42 OK

- Check minimum limit of reinforcement: (LRFD Art. 5.7.3.3.1)

ρ_{act} = $1.056 / (12 \times 6.125)$ = 14.36×10^{-3}

$\rho_{min.}$ = $0.03\ (f_c^{'} / f_y)$ = 0.03x4.0/60 = 2.00×10^{-3}

< ρ_{act} OK

- Check of crack control by distribution of reinforcement: (LRFD Art. 5.7.3.4)

d_c = (2.5 + 0.5 x 0.75) = 2.875 in.

A_c = (2 x 2.875 x 5) = 28.75 in^2

Z = 170 kip/in

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = \frac{170}{\sqrt[3]{28.75 \times 2.875}} = 39.0\ \text{ksi} > (0.6\ f_y = 36\ \text{ksi})$$

$f_{sa\ all.}$ = 36 ksi

Actual stress in reinforcement:

E_c = $33{,}000\ (w_c)^{1.5} \sqrt{f_c^{'}}$ = 3835 ksi

n = E_s / E_c = 7.56

ρ_{act} = 14.36×10^{-3}

k = $\sqrt{(\rho n)^2 + (2\rho n)} - \rho n$ = 0.478

j = (1 - k/3) = 0.841

$f_{sa\ act.}$ = $M_{service} / (jdA_s)$

= (8.7x12)/(0.841x6.125x1.056)= 19.2 ksi

< $f_{sa\ all.}$ OK

- Check fatigue limit state: (LRFD Art. 5.5.3)

Fatigue need not to be investigatedfor concrete deck slabs in multi-girder applications.

Check development length of steel reinforcement at critical section over the exterior girders (LRFD Art. 5.11.2)

$$l_d = \frac{1.25\phi_b f_y}{\sqrt{f_c^{'}}} = \frac{1.25 \times 0.75 \times 60}{\sqrt{4.0}} = 28.1\ \text{in.} < l_{act.} = 3.75\ \text{ft} \quad \text{OK}$$

Distribution reinforcement (LRFD Art. 9.7.3.2)

$$\rho = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{11.5x12}} = 65\ \%$$

Distribution reinforcement = $\rho\ (A_s)$ = 0.65x0.53 = 0.35 in.^{2} /ft

Use #4 @ 6 in, therefore, A_s provided = 0.20x12/6 = 0.44 in^2 OK

9 inch CIP deck reinforced with epoxy coated WWF

Design assumptions:

A 44.0 ft-wide bridge deck supported by 4 steel girders spaced at 12 ft and has two overhangs each of 4.0 ft length is considered.

Girder spacing = 12.0 ft

Overhang length = 4.0 ft

A 12 in. wide flange steel girders are considered as supporting elements of the slab

Design according to AASHTO Standard Specification, 15th edition.

Specified concrete strength at time of opening the bridge for traffic = 4.0 ksi

Welded wire fabric reinforcement :

Yield strength = 60 ksi

Modulus of elasticity = 29,000 ksi

Top reinforcement clear cover = 2.5 in.

Bottom reinforcement clear cover = 1.0 in.

A 1/2 in. wearing surface is considered to be an integral part of the 9 in. slab

2 in. concrete future wearing surface is considered

HS-25 loading (HS-20 AASHTO Live loading modified by a factor of 5/4)

Minimum slab thickness: (AASHTO Art. 8.11.1)

AASHTO Standard does not specify minimum slab thickness for the case of having the main reinforcement perpindicular to the traffic direction. It recommends only a minimum thickness for reinforced concrete slab for the case of main reinforcement parallel to the traffic (Table 8.9.2). However, minimum thickness provision is checked.

S = (Clear distance between edges of flanges) + 0.5(Flange width)

= 11.0 ' + 0.5(1.0 ') = 11.5 ft

$t_{min.} = \dfrac{S+10}{30} = 0.72 \text{ ft} = 8.64 \text{ in.}$

Use 9 in. slab thickness.

Loads:

Dead loads (DL):	Self weight of 9" CIP slab	= (9/12) (0.150)	= 0.113 kip/ft
	Railing weight, see Fig. F-3.		= 0.382 kip/ft
	Self weight of 2" wearing surface	= (2/12) (0.150)	= 0.025 kip/ft
Live loads (LL):	HS-25 Loading with impact.		

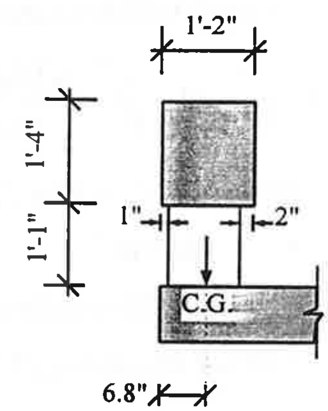


Fig. F-3. Railing Dimensions

Load Combination: (AASHTO Art. 3.4)

$$U = 1.3\,[(DL) + 1.67(L+I)\,]$$

Design of section of positive moment section (between girders):

- Bending moment:

DL: Due to own weight of slab, $M_s = 0.113\,(11.5^2/10) = 1.494$ ft-kips

Due to wearing surface, $M_{ws} = 0.025\,(11.5^2/10) = 0.331$ ft-kips

LL: Impact: (AASHTO Eq. 3-1)

$I = 50 / (L+125) = 50/(11.5 + 125) = 0.37 > 0.3$

use I = 30%

For HS-20, $M_{LL} = \left(\dfrac{S+2}{32}\right)P_{20}$ (AASHTO Eq. 3-15)

Thus, for HS-25, 0.30 impact, and 0.8 continuity factor (Art. 3.24.3.1):

$M_{LL} = 1.25\left(\frac{11.5+2}{32}\right)(16)\ (1+0.30)(0.8) =$ 8.775 ft-kips

Total service bending moment = 1.494 + 0.331 + 8.775 = 10.600 ft-kips

Total factored bending moment = 1.3(1.494 + 0.331 + 1.67x8.775)

= 21.423 ft-kips

- Design of section:

Try welded wire fabric D22. Thus, with 1.0 in. clear cover, and 0.5 in. sacrificial surface:

$d = 9 - 0.5x0.529 - 1 - 0.5 =$ 7.24 in.

$R_n = (M_u / 0.9bd^2) = (26.19x12x1000)/(0.9x12x7.24^2) =$ 555 lb/in^2

$m = (f_y / 0.85f_c') = (60{,}000)/(0.85x4{,}000) =$ 17.65

$$\rho = \frac{1}{m}(1-\sqrt{1-2\frac{mR_n}{f_y}}) = \frac{1}{17.65}(1-\sqrt{1-\frac{2x17.65x555}{60{,}000}}) = 10.16 \times 10^{-3}$$

$A_s = \rho\ (bd) = 10.16 \times 10^{-3} \times 12 \times 7.24 =$ 0.88 in.^{2}

Use W22 @ 4 in., A_s = 0.88 in.^{2} OK

$a = (A_s f_y) / (0.85\ b\ f_c') = (0.88x60{,}000)/(0.85x12x4{,}000) =$ 1.29 in.

$\varphi M_n = 0.9\ (A_s f_y)\ (d - a/2) = (0.9x0.88x60{,}000)(7.24-0.5x1.29)/(12x1000)$

= 26.13 ft-kips > M_u OK

Due to manufacture limitation, W31@3 in. was used which gives φM_n =42.9 ft-kips

- Check minimum reinforcement: (AASHTO Art. 8.17.1)

$1.2M_{cr} = 1.2(1.25\ bH^2 \sqrt{f_c'}) = 1.2 \times 1.25 \times 12 \times 9^2 \sqrt{4000} / (12000) =$ 7.6 ft-kips

< φM_n OK

- Check crack control by distribution of reinforcement: (AASHTO Art. 8.16.8.4)

$d_c = (1 + 0.5 \times 0.529) =$ 1.265 in.

$A_c = (2 \times 1.265 \times 3) =$ 7.59 in.^{2}

$Z =$ 170 kip/in

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = 80.0 \text{ ksi} > (0.6\ f_y = 36 \text{ ksi})$$

$f_{sa\ all.} =$ 36 ksi

Actual stress in reinforcement:

E_c	=	$33\,(w_c)^{1.5}\,\sqrt{f_c^{'}}$ =	$33\,(150)^{1.5}\,\sqrt{4{,}000}$	=	3835	ksi	
n	=	E_s / E_c =	29,000/3835	=	7.56		
ρ_{act}	=	0.88 / (12x7.24)	=	$10.13\text{x}10^{-3}$			
k	=	$\sqrt{(\rho n)^2 + (2\rho n)}$	=	0.400			
j	=	(1 - k/3)	=	0.867			
$f_{sa\ act.}$	=	$M_{service} / (jdA_s)$	=	23.0	ksi	<	$f_{sa\ all.}$ OK

Design of section of negative moment over interior girder lines

- Bending moment:

DL: Due to own weight of slab, $M_s = 0.113\ (11.5^2/10)$ = 1.494 ft-kips

Due to wearing surface, $M_{ws} = 0.025\ (11.5^2/10)$ = 0.331 ft-kips

LL: Impact: $I = 30\%$ (AASHTO Eq. 3-1)

For HS-20, $M_{LL} = \left(\frac{S+2}{32}\right)P_{20}$ (AASHTO Eq. 3-15)

Thus, for HS-25, 0.30 impact, and 0.8 continuity factor (Art. 3.24.3.1):

$M_{LL} = 1.25\left(\frac{11.5+2}{32}\right)(16)\,(1+0.30)(0.8) =$ 8.775 ft-kips

Total service bending moment = 1.494 + 0.331 + 8.775 = 10.600 ft-kips

Total factored bending moment, M_u = 1.3(1.494 + 0.331 + 1.67x8.775)

= 21.423 ft-kips

- Design of section:

Assume using D31, thus with 2.5 in. clear cover, therefore:

d	= 9 - 0.5x0.628 - 2.5 =	6.186	in.
b	= =	12.0	in.
R_n	$= (M_u / \phi b d^2)$ =	622	lb/in^2, $(\phi = 0.9)$
m	$= (f_y / 0.85 f_c^{'})$ =	17.65	
ρ	$= \frac{1}{m}\left(1-\sqrt{1-2\frac{mR_n}{f_y}}\right) =$	11.54×10^{-3}	
A_s	$= \rho\,(bd)$ =	$0.857\ \text{in}^2$	
Use W31 @ 4 in., A_s	=	$0.93\ \text{in}^2$	

$a \quad = (A_s f_y) / (0.85 \ b \ f'_c) \ = \quad 1.37$ in.

$\varphi M_n \quad = 0.9 \ (A_s f_y) \ (d - 0.5a) \ = \quad 23.022$ ft-kips $> \quad M_u$ OK

- Check minimum reinforcement: (AASHTO Art. 8.17.1)

$1.2M_{cr} = 1.2(1.25 \ bH^2 \sqrt{f'_c}\,) = 1.2 \times 1.25 \times 12 \times 9^2 \sqrt{4000} / (12{,}000) = 7.6$ ft-kips

$< \quad \phi M_n$ OK

- Check crack control for distribution of reinforcement: (AASHTO Art. 8.16.8.4)

$d_c \quad = (2.5 + 0.5 \times 0.628) = \quad 2.814$ in.

$A_c \quad = (2 \times 2.814 \times 3) \ = \quad 16.88 \ in^2$

$Z \quad = \quad 170$ kip/in.

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = 47 \text{ ksi} > (0.6 f_y = 36 \text{ ksi})$$

$f_{sa\ all.} = 36$ ksi

Actual stress in reinforcement:

$E_c \quad = \quad 33 \ (w_c)^{1.5} \sqrt{f'_c} \quad = \quad 3835$ ksi

$n \quad = \quad E_s / E_c \quad = \quad 7.56$

$\rho_{act} \quad = \quad 1.24 / (12 \text{x} 6.186) \quad = \quad 16.70 \times 10^{-3}$

$k \quad = \quad \sqrt{(\rho n)^2 + (2\rho n)} - \rho n \quad = \quad 0.52$

$j \quad = \quad (1 - k/3) \quad = \quad 0.83$

$f_{sa\ act.} \quad = \quad M_{service} / (jdA_s) \quad = \quad 24.1$ ksi $< \quad f_{sa\ all.}$

OK

Design of section of negative moment over exterior girder line:

- Bending moment:

Assume wheel load applied at 1 ft from face of railing, as shown in Fig. F-4.

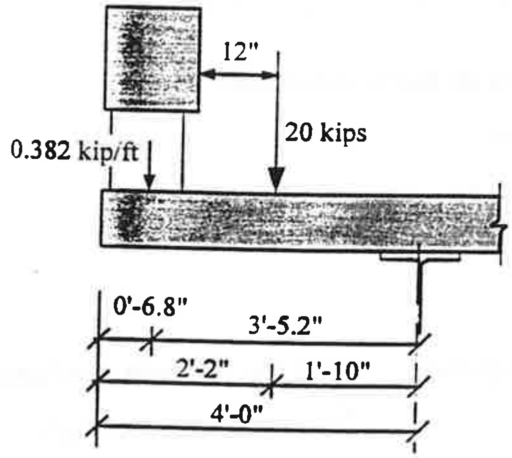


Fig. F-4. HS-25 Loading for the Overhang

$M_{D\,(slab+ws)}$ = $(0.113+0.025)\,(3.75^2)/2$ = 0.970 ft-kips

$M_{D\,(barrier)}$ = $0.382\,(3.433-0.25)$ = 1.216 ft-kips

Live load: wheel load is distributed over a width of (AASHTO Art. 3.24.5)

$E = 0.8\,X + 3.75 = 0.8(1.833-0.25) + 3.75$ = 5.02 ft

$M_{L+I} = 1.30 \times 20 \times (1.833-0.25)/5.02$ = 8.199 ft-kips

Total service bending moment = 10.385 ft-kips

Total factored bending moment = 20.606 ft-kips

These values are less than those used to design negative moment section over interior girder lines. Thus use same reinforcement

Check punching shear: (AASHTO Art. 3.30)

Tire contact area = 0.01 P = 0.01 x 20,000 = 200 in^2

= b x w

and w = 2.5 b

Therefore, b = 8.94 in., w = 22.36 in.

Minimum d is at the overhang = 6.186 in.

$b_o = 2\,[\,(b+d) + (w+d)\,]$ = 87.344 in.

$V_{c\,provided} = (0.8 + \frac{2}{2.5})\sqrt{f'_c}\; b_o d$ = 54.7 kips

V_u = (Wheel load x Impact) = 20 x 1.33 = 26 kips $< V_{c\,provided}$ OK

Distribution reinforcement: (AASHTO Art. 8.20)

$A_{s\,min.}$ = (1/8) in.^{2}/ ft = 0.125 in.^{2}/ ft

Use W 18 @ 6", thus A_s provided = 0.36 in.^{2}/ ft

Conventional Precast Deck Sub-Panel System

Design assumptions:

AASHTO Standard Specification, 15th edition.

Girder spacing = 12.0 ft

Overhang length = 4.0 ft

Precast prestressed panel: Concrete strength at release = 4.0 ksi

28-day concrete strength = 10.0 ksi

0.5 in. indented diameter strand, 270 ksi, low relaxation

Modulus of elasticity = 29,000 ksi

Panel dimensions:

(width x length x thickness) = (3'-9" x 11'-6" x 3")

Cast-in-place concrete: Specified concrete strength at time of opening the bridge for traffic = 4.0 ksi

Reinforcement is welded wire fabric:

yield strength = 60 ksi

Modulus of elasticity = 29,000 ksi

Top reinforcement clear cover = 2.5 in.

Bottom reinforcement clear cover = 1.0 in.

A 12 in. wide flange steel girders are considered as supporting elements of the slab

HS-25 loading (HS-20 AASHTO Live loading modified by a factor of 5/4)

Minimum slab thickness: (AASHTO Art. 8.11.1)

S = (Clear distance between edges of flanges) + 0.5(Flange width)

$= 11.0' + 0.5(1.0') = 11.5$ ft

$$t_{min.} = \frac{S+10}{30} = 0.72 \text{ ft} = 8.64 \text{ in.}$$

Use 9 in. Slab thickness.

Loads:

Dead loads:

self weight of 3 in. SIP panel =	(3/12) (0.150) =		0.0375 kip/ft
self weight of 6 in. CIP slab =	(6/12) (0.150) =		0.075 kip/ft
self weight of 2 in. wearing surface =	(2/12)(0.150) =		0.025 kip/ft
Construction load =			0.050 kip/ft

Live loads: HS-25 loading with impact.

Impact, I $= 50 / (L+125) < 0.3$ (AASHTO Eq. 3.1)

$= 50/(11.5 + 125) = 0.37 > 0.3$

use I = 30%

Note: SIP panel and CIP slab self weight act on the noncomposite section, while the wearing surface and the live loads act on the composite section.

Section properties:

Panel dimensions: (3 in. x 11.5 ft x 3.75 ft)

- for the non-composite section:

$A = 3 \times 12 = 36 \text{ in.}^2/\text{ft}$

$Z_t = Z_b = (12 \times 3^2)/6 = 18 \text{ in.}^4/\text{ ft}$

$E_{ci} = 33(150)^{1.5}\sqrt{4{,}000} = 3834 \text{ ksi}$

$E_c = 33(150)^{1.5}\sqrt{10{,}000} = 6062 \text{ ksi}$

- For the composite section:

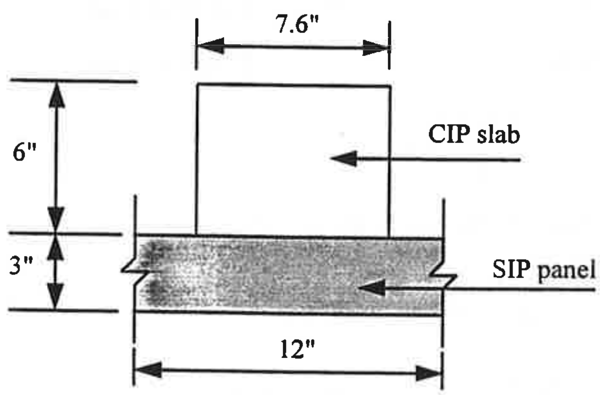


Fig. F-5 Transformed Section

$E_{precast} = 33(150)^{1.5}\sqrt{10{,}000}$

$E_{CIP} = 33(150)^{1.5}\sqrt{4{,}000}$

n	=	$E_{CIP} / E_{precast}$			=	0.632	
A_c	=	12x3 + 12x0.632x6	=	36 + 45.6	=	81.6	in.^{2}
Y_{bc}	=	(36x1.5 + 45.6x6.0)/(81.6)			=	4.01	in.
$Y_{tc\ SIP}$	=	4.01 - 3.0			=	1.01	in.
$Y_{tc\ CIP}$	=	9 - 4.01			=	4.99	in.
I_c	=	$(36\text{x}3^2/12) + 36(4.01-1.5)^2 + (45.6\text{x}6^2/12) + (45.6)(6-4.01)^2$					
					=	571	in.^{4}
$Z_{tc\ CIP}$	=	571/4.99			=	114.4	in.^{3}
$Z_{tc\ SIP}$	=	571/1.01			=	565.4	in.^{3}
Z_{bc}	=	571/4.01			=	142.4	in.^{3}

Prestressing strands:

Estimation of the required number of strands is usually governed by concrete tesile stress at service loads.

Bottom concrete stresses due to SIP panel self-weight, CIP concrete, wearing surface, and live loads is

$$f_b = \frac{\cancel{M}_{SIP} + M_{CIP}}{Z_b} + \frac{M_{ws} + M_{LL}}{Z_{bc}}$$

$M_{SIP} = 0.0375\text{x}11.5^2/8 \quad = \quad 0.620$ ft-kips

$M_{CIP} = 0.0750\text{x}11.5^2/8 \quad = \quad 1.240$ ft-kips

$M_{ws} = 0.0250\text{x}12.0^2/10 \quad = \quad 0.360$ ft-kips

For HS-20, $M = \left(\frac{S+2}{32}\right)P_{20}$

Thus for HS-25, impact factor of 0.30, and 0.8 continuity factor:

$$M_{LL} = 0.8\left(\frac{11.5+2}{32}\right)(16\text{x}1.25)(1+0.3) = 8.775 \text{ ft-kips}$$

$$f_b = \frac{(0.620+1.240)12}{18} + \frac{(0.360+8.775)12}{142.4}$$

$$= 1.240 + 0.770 = 2.010 \text{ ksi}$$

Allowable tensile concrete stress at service loads = $6\sqrt{f'_{ci}} = 6\sqrt{10{,}000}/1000 = 0.600$ ksi

Recent research showed that allowable tensile concrete stress at service loads of high strength concrete can be raised up to $8\sqrt{f'_{ci}}$.

Take allowable tensile concrete stress at service loads

$$= 7.5\sqrt{f'_{ci}} = 7.5\sqrt{10{,}000}/1000 = 0.750 \text{ ksi}$$

Required precompression stress at bottom fiber = 2.010 - 0.750 = 1.260 ksi

If P_{se} is the total effective prestress force after all losses, and the ceter of gravity of strands is concentric with the center of gravity of the SIP panel, thus

$$1.260 = \frac{p_{se}}{A} = \frac{p_{se}}{36}$$

P_{se} = 45.4 kips/ft = 45.4x3.75 = 170.3 kips/panel

Final prestress force per strand assuming 1/2 in. diameter strand and 20% final losses

$= A_p\, f_{pi}$ (1-final losses%)

= (0.153)(0.75x270)(1-20%) = 24.8 kips

Required number of strands = 170.3 / 24.8 = 6.9

Try 7-0.5 in. diameter, 270 ksi, low relaxation strands.

Prestress losses: (AASHTO Art. 9.16)

- Shrinkage:

SH = 17,000 - 150(RH)

= [17,000 - 150(70)] / 1000 = 6.5 ksi

- Elastic shortening:

E_{strand} = 29,000 ksi

$E_{precast} = 33(150)^{1.5}\sqrt{4{,}000}/1000$ = 3,834 ksi

f_{cir} = stress at the center of gravity of prestressing force due to prestressing force and the own weight of the panel immediately after transfer

Assume an initial prestress loss of 8%, thus total prestress force after release

= (0.153)(0.75x270)(1-8%)(7 strands) = 199.5 kips

f_{cir} = 199.5 / (3x45) = 1.478 ksi

$ES = (E_s f_{cir} / E_{ci}) = (29{,}000 \times 1.478)/ 3{,}834 = 11.2$ ksi

- Creep:

$f_{cir} = 1.478$ ksi

f_{cds} = concrete stress at the center of gravity of prestressing steel due to all dead loads except the dead load present at the time of applying the prestressing force, (due to self weight of CIP slab and wearing surface) = 0.022 ksi

$M_{CIP} = 0.0750 \times 11.5^2/8 = 1.240$ ft-kips (acts on the noncomposite section)

$M_{ws} = 0.0250 \times 12.0^2/10 = 0.360$ ft-kips (acts on the composite section)

However, the CIP slab provide zero concrete stress at the center of gravity of prestressing force. Thus, stress due to wearing surface is only considered as shown in Fig. F-6.

Top fiber CIP slab concrete stress = $0.360 \times 12 / 114.4 = 0.038$ ksi

Bottom fiber SIP panel concrete stress= $0.360 \times 12 / 142.4 = 0.030$ ksi

Thus, $f_{cds} = 0.030 \times 2.51 / 4.01 = 0.019$ ksi

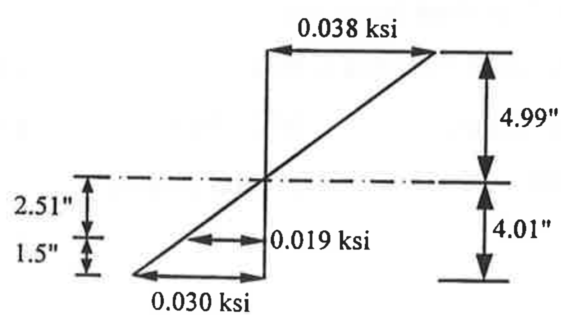


Fig. F-6. Concrete Stresses due to Wearing Surface

$CR_c = 12 f_{cir} - 7 f_{cds} = 12 \times 1.478 - 7 \times 0.019 = 17.6$ ksi

- Relaxation of prestressing steel:

$CR_s = 5000 - 0.1(ES) - 0.05(SH+CR_c)$

$= 5.0 - 0.1 \times 11.2 - 0.05(6.5+17.6) = 2.7$ ksi

- Initial loss = ES = = 11.2 ksi

Initial prestress loss = $11.2 / (0.75 \times 270)$ = 5.5 %

5.5% initial loss is close the first estimation (8%), thus there is no need to go through a second iteration to refine initial loss.

Total prestress force at release $= (0.153)(0.75x270)(1.-5.5\%)(7 \text{ strands})$

$= 204.9$ kips

$= 204.9 / 3.75 =$ 54.6 kips/ft

- Final losses $= 6.5 + 11.2 + 17.6 + 2.7 =$ 38.0 ksi

Final prestress loss $=$ 38.0 / 202.5 $=$ 19 %

19% final loss is close the first estimation (20%), thus there is no need to go through a second iteration to refine final loss.

Total prestress force at service loads $= (0.153)(0.75x270)(1.-19\%)(7 \text{ strands})$

$= 175.7$ kips

$= 175.7 / 3.75 =$ 46.9 kips/ft

Check stresses at release:

- Sign conventions: concrete compresive stress is positive
  concrete tensile stree is negative
- Allowable stresses: (AASHTO Art. 9.15)

  Compression $=$ $0.6\, f_{ci}' = 0.6 \times 4.000 =$ +2.4 ksi

  Tension:

  with no bonded reinforcement $= 3\sqrt{f_{ci}'} \leq 200$ ksi

  $= -3\sqrt{4000} / 1000 = -0.189$ ksi > -0.200 ksi OK

  with bonded reinforcement $= 7.5\sqrt{f_{ci}'}$

  $= -7.5\sqrt{4000} / 1000 = -0.474$ ksi

Effective prestressing stress at the end of prestressing process $=$ 54.6 kips/ft

Bending moment due to self weight of the panel $= 0.0375 \times 12^2 / 8 =$ 0.675 ft-kips

Top concrete stress of the SIP panel, $f_{t\,SIP} = +(54.6/36) + (0.675x12/18)$

$= +1.517 + 0.450 =$ +1.967 ksi

Allowable compresive concrete stress is 2.4 ksi. OK

Bottom concrete stress of the SIP panel, $f_{b\,SIP} = +(54.6/36) - (0.675x12/18)$

$= +1.517 - 0.450 =$ +1.067 ksi

Allowable compresive concrete stress is 2.4 ksi. OK

Check stresses at time of casting top slab:

Assume that, at time of casting the CIP concrete slab, the precast SIP concrete panel reaches its 28-day concrete strength (10 ksi).

- Allowable stresses: (AASHTO Art. 9.15.2.2)

Compression:

due to all load combinations

$$= \quad 0.6\ f_c^{'} = 0.6 \text{ x } 10{,}000 / 1000 = \quad +6.0 \text{ ksi}$$

due to effective prestress and permanent (dead)loads

$$= \quad 0.6\ f_c^{'} = 0.4 \text{ x } 10{,}000 / 1000 = \quad +4.0 \text{ ksi}$$

Tension:

for members with bonded reinforcement $= \quad 6\sqrt{f_c^{'}}$

$$= \quad -6\sqrt{10{,}000} / 1000 \quad = \quad -0.6 \text{ ksi}$$

Recent research with high performance concrete showed that allowable concrete stress can be raised up to $7.5\sqrt{f_c^{'}} \quad = \quad -7.5\sqrt{10{,}000} / 1000 \quad = \quad -0.75$ ksi

Effective prestressing stress aftre all losses = 46.9 kips/ft

Bending moment due to SIP panel self weight, CIP concrete self weight, and construction load are

$$M_{SIP} = (0.0375)\,(11.5^2) / 8 = 0.620 \text{ ft-kips}$$

$$M_{CIP} = (0.075)\,(11.5^2) / 8 = 1.240 \text{ ft-kips}$$

$$M_{const} = (0.05)\,(11.5^2) / 8 = 0.827 \text{ ft-kips}$$

$$M_{total} = 2.687 \text{ ft-kips}$$

Top concrete stress of the SIP panel, $f_{t\ SIP}$ $= +(46.9/36) + (2.687 \text{ x}12/18)$

$$= +1.303 + 1.791 = \quad +3.094 \text{ ksi}$$

Allowable compresive concrete stress is 6.0 ksi OK

Bottom concrete stress of the SIP panel, $f_{b\ SIP} = +(46.9/36) - (2.687 \text{ x}12/18)$

$$= +1.303 - 1.791 = \quad -0.488 \text{ ksi}$$

Allowable tensile concrete stress is -0.75 ksi OK

Check stresses at time of opening the bridge for traffic:

- Allowable stresses:

Compression:

for CIP concrete slab = $0.6\, f_{ci}^{'} = 0.6 \times 4.000$ = +2.4 ksi

for precast concrete:

due to all load combinations

= $0.6\, f_c^{'} = 0.6 \times 10{,}000 / 1000$ = +6.0 ksi

due to effective prestress and permanent (dead)loads

= $0.6\, f_c^{'} = 0.4 \times 10{,}000 / 1000$ = +4.0 ksi

Tension for precast concrete:

for members with bonded reinforcement $= 6\sqrt{f_c^{'}}$

= $-6\sqrt{10{,}000} / 1000$ = -0.6 ksi

Recent research with high performance concrete showed that allowable concrete stress can be raised up to $7.5\sqrt{f_c^{'}}$ = $-7.5\sqrt{10{,}000} / 1000$ = -0.75 ksi

- Effective prestressing stress aftre all losses = 46.9 kips/ft

SIP panel self weight and CIP concrete self weight act on noncomposite section, thus

M_{SIP} = $(0.0375)\,(11.5^2) / 8$ = 0.620 ft-kips

M_{CIP} = $(0.075)\,(11.5^2) / 8$ = 1.240 ft-kips

At time of opening the bridge to traffic, wearing surface load and the live load act on the composite section.

Bending moment due to wearing surface load is

$M_{ws} = 0.025 \times 12^2 / 10$ = 0.360 ft-kips

For HS-20, M_{LL} = $\left(\dfrac{S+2}{32}\right) P_{20}$ (AASHTO Art. 3.24.3.1)

Thus, for HS-25, impact factor of 0.30, and 0.8 continuity factor:

M_{L+I} = $0.8\left(\dfrac{11.5+2}{32}\right)(16\text{x}1.25)(1+0.3)$ = 8.775 ft-kips

Therefore:

Concrete stress at top fiber of the CIP slab, $f_{t\,CIP}$ $= +0.632 \dfrac{(0.360 + 8.775)12}{114.4}$

$= +0.606$ ksi

Allowable compresive concrete stress is 2.4 ksi OK

Concrete stress at top fiber of the SIP slab,

$$f_{t\,SIP} = +\frac{(46.9)12}{36} + \frac{(0.620 + 1.240)12}{18} - \frac{(0.360 + 8.775)12}{565.4}$$

$$= +1.303 + 1.240 - 0.194 = +2.349 \text{ ksi}$$

Allowable compresive concrete stress is 6.0 ksi OK

Concrete stress at bottom fiber of the SIP slab,

$$f_{t\,SIP} = +\frac{(46.9)12}{36} - \frac{(0.620 + 1.240)12}{18} - \frac{(0.360 + 8.775)12}{142.4}$$

$$= +1.303 - 1.240 - 0.770 = -0.707 \text{ ksi}$$

Allowable tensile concrete stress is -0.75 ksi OK

Flexural strength at first interior maximum positive moment section:

(AASHTO Art. 9.17)

$M_{SIP} = (0.0375)(11.5^2)/8 = 0.620$ ft-kips

$M_{CIP} = (0.075)(11.5^2)/8 = 1.240$ ft-kips

$M_{ws} = (0.025)(12^2)/10 = 0.360$ ft-kips

$M_{L+1} = 8.775$ ft-kips

$M_u = 1.3\,[\,M_D + 1.67\,M_{L+I}\,]$

$= 1.3\,[\,(0.620+1.240+0.360) + 1.67(8.775)\,] =$

$= 21.9$ ft-kips

$A_s = (7 \times 0.153)/(3.75) = 0.2856$ in.^{2}/ft

$d = 1.5 + 6 = 7.5$ in.

$\rho = 0.2856/(7.5 \times 12) = 0.00317$

f_c' (CIP concrete) $= 4.0$ ksi

$f_s' = 270$ ksi

$$f^*_{su} = f_s'\left(1 - 0.5\frac{\rho^* f_s'}{f_c'}\right)$$

$$= 270\left(1 - \frac{0.5x0.00317x270}{4.0}\right) = 241.1 \text{ ksi}$$

Check f^*_{su} : (AASHTO Eq. 9-19)

$$f^*_{su} \leq \frac{L_x}{D} + \frac{2}{3} f_{se}$$

L_x = 11.5 / 2 = 5.75 ft , D = 0.5 in. , f_{se} = (0.75x270 - 38.0) = 164.5 ksi

$$\frac{L_x}{D} + \frac{2}{3} f_{se} = (5.75x12/0.5 + 2x164.5/3) = 247.7 \text{ ksi}$$

Therefore: f^*_{su} = 241.1 ksi (Controls)

$\varphi M_n = \varphi A_s f^*_{su} d (1 - 0.6 \rho f^*_{su} / f_c')$

= 1.0x0.2856x241.1x7.5 (1 - 0.6x0.00317x241.1/4.0)

= 38.1 ft-kips /ft $> M_u$ OK

Check ductility limit: (AASHTO Art. 9.18)

Check maximum prestressing steel:

$(\rho f^*_{su} / f_c') = (0.00317 \text{ x } 241.1 / 4.0) = 0.19 < 0.3$ OK

Check minimum steel:

$$M_{cr}^* = (f_r + f_{pe})S_c - (M_{d/nc})(S_c / S_b - 1)$$

$f_r = 7.5\sqrt{f_c'} = 7.5\sqrt{10{,}000}/1000$ = 0.750 ksi

f_{pe} = 46.9/36 = 1.303 ksi

$M_{d/nc} = M_{SIP} + M_{CIP}$ = 0.620 + 1.240 = 1.860 ksi

$M_{cr}^* = (0.750 + 1.303)(142.4 / 12) - (1.860)(142.4 / 18 - 1)$ = 11.5 ft-kips

$1.2\ M_{cr}^*$ = 13.8 ft-kips $\leq \varphi M_n$ OK

Distribution reinforcement steel in deck panel: (AASHTO Art. 9.23.2)

$A_{S\,min.}$ = 0.11 in^2./ ft

Use 4x4 D5xD5, therefore A_s = 0.15 in.^{2}

Design of the negative moment section:

The same as in section earlier.

Continuous Precast Deck Sub-Panel System

Design assumptions:

- AASHTO Standard Specifications.
- Precast prestressed panel:

Concrete: Specified concrete strength at release = 4.0 ksi

Specified 28-day compressive strength = 10.0 ksi

Prestressing reinforcement:

0.5 in. diameter indented strand, 270 ksi, low relaxation, ASTM-A421.

Ultimate strength	f'_s		= 270 ksi
Yield strength	f^*_y	$= 0.9 f'_s$	= 243 ksi (AASHTO Art. 9.15)
Initial prestressing	f_{si}	$= 0.75 f'_s$	= 202.5 ksi (AASHTO Art. 9.15.1)
Modulus of elasticity	E_s		= 28,000 ksi (AASHTO Art. 9.16.2.1.2)

Non-prestressing reinforcement:

ASTM-A616, deformed bars, black steel

Yield strength = 60 ksi

Modulus of elasticity = 29,000 ksi (AASHTO Art. 8.7.2)

Mean relative humidity = 70%

- Cast-in-place concrete:

Concrete: compressive strength at time of opening the bridge for traffic = 4.0 ksi

Reinforcement: welded wire fabric:

yield strength	=	60 ksi
Modulus of elasticity	=	29,000 ksi
Top reinforcement clear cover	=	2.5 in.
Bottom reinforcement clear cover	=	N.A.

- Girder spacing = 12 ft
- 1/2 in. of the CIP topping is considered as an integral wearing surface.
- HS-25 loading, equivalent to HS-20 AASHTO loading magnified with a factor of 1.25, is considered.
- Open the bridge for traffic when the CIP deck slab has a compressive strength of 4000 ksi or more.
- Allowance for a future concrete wearing surface of 2 inch was considered.
- No loads should be provided on the overhangs of the SIP panels before the mortar mix, filling the gaps over the girder lines, get its specified strength, 4,000 psi.
- SIP panel is designed to resist the loads of topping slab self weight, a construction load of 50 lb/ft^2, and concrete paving machine weight.
- Composite section is designed to resist the superimposed loads which are the future wearing surface self weight, barrier weight, and live load.

Recommended minimum slab thickness: (AASHTO Art. 8.11.1)

AASHTO Standard does not specify minimum slab thickness for the case of using SIP prestressed panel with CIP topping. It recommends only a minimum thickness for reinforced concrete slab for the case of main reinforcement parallel to the traffic (Table 8.9.2). Although the main reinforcement in the proposed system is perpendicular to the traffic, minimum thickness provision is checked.

S = (Clear distance between edges of flanges) + 0.5(Flange width) =

= 11.0 ' + 0.5(1.0 ') = 11.5 ft

$$t_{mim.} = \frac{S+10}{30} = 0.72 \text{ ft} = 8.64 \text{ in.}$$

Total thickness provided = 4.5 + 4.5 = 9 in. > $t_{min.}$ OK

Therefore deflection does not need to be checked such that deflection due to service load plus impact shall not exceed (1/800) for continuous spans and (1/300) for cantilevers. (Art. 8.9.3.1).

Article 9.7.1.1 in AASHTO LRFD specifies that the depth of the concrete deck excluding any provisions for grinding grooving and sacrificial surface should not be less than 7.0 in. Also, Article 9.7.4.3 in AASHTO LRFD specifies that the depth of the SIP concrete should neither exceed 55% of the depth of the finished deck slab nor be less than 3.5 in. Both conditions are satisfied with the proposed system.

Loads:

Dead loads:	4.5 in. SIP panel self weight	= (4.5/12) (0.150) =	0.0563 kip/ft
	4.5 in. CIP topping self weight	= (4.5/12) (0.150) =	0.0563 kip/ft
	2 in. concrete wearing surface	= (2/12)(0.150) =	0.025 kip/ft
	Construction load	=	0.050 kip/ft

Live loads: HS-25 loading, equivalent to HS-20 AASHTO Standard truck magnified with a factor of 1.25.

Impact, I = 50 / (L+125) (AASHTO Eq. 3.1)

= 50/(11.5 +125) = 0.37 > 0.3

use I = 30%

Concrete paving machine: Loads, given in Fig. F-7., are maximum reactions when screed head is at the respective support location. These reactions are for 30 ft wide deck machine. For each extra length of 1 ft add 60 lbs. Total weight of 30 ft wide deck machine is 6674 lbs.

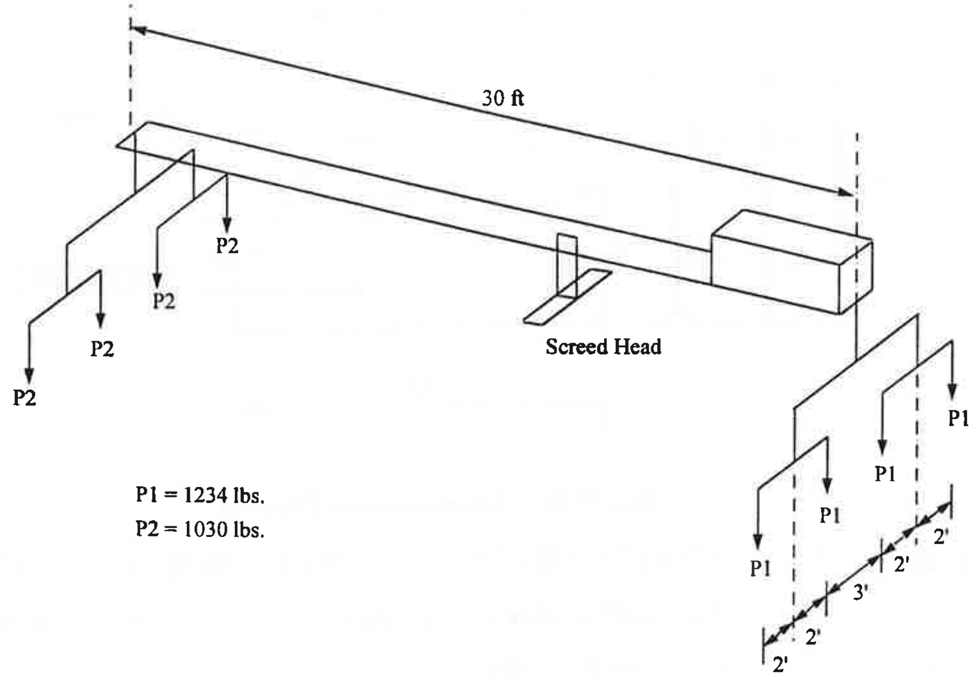


Fig. F-7. Paving Macchine Loads

Section properties:

- for the non-composite section:

A	$= 4.5 \times 12$	=	54 in^2/ft
Z_{top}	$= Z_{bott} = (12 \times 4.5^2)/6$	=	40.5 in^3/ft
E_{ci}	$= 33(150)^{1.5}\sqrt{4000}$	=	3834 ksi

- For the composite section:

The composite properties should be calculated considering deck slab compressive strength at time of opening the bridge for traffic is 4000 ksi.

$E_{precast} = 33(150)^{1.5}\sqrt{10,000}$

$E_{CIP} = 33(150)^{1.5}\sqrt{4,000}$

$n = E_{CIP} / E_{precast} = 0.63$

Transformed width of the topping concrete = 12×0.63 = 7.6 in.

Fig. F-8. gives the dimensions of the transformed composite section. Note that 1/2 inch from the CIP topping is considered as an integral wearing surface.

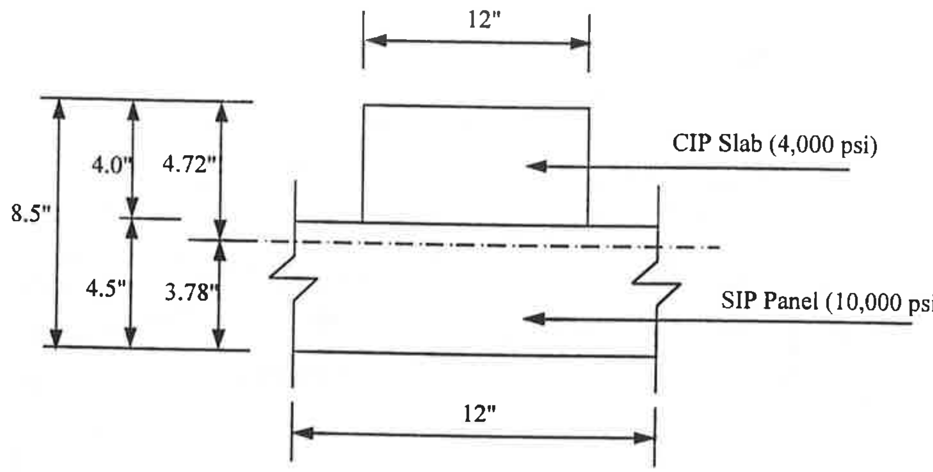


Fig. F-8. Transformed Section

$A_{transformed}$ = (4.5 x 12 + 4.0 x 7.6) = (54.0 + 30.4) = 84.4 in^2

Y_{bott} = (54 x 2.25 + 30.4 x 6.5) / 84.4 = 3.78 in.

$Y_{bott.\ CIP}$ = (4.5 - 3.78) = 0.72 in.

$Y_{top\ CIP}$ = (4.5 + 4.0 - 3.78) = 4.72 in.

I = $(54 \times 4.5^2 / 12) + (54)(3.78 - 2.25)^2$

$+ (30.4 \times 4.0^2 / 12) + (30.4)(4.72 - 2.0)^2$ = 483.0 $in.^4$

$Z_{bott.}$ = 483.0 / 3.78 = 127.8 $in.^3$

$Z_{bott.\ CIP}$ = 483.0 / 0.72 = 670.8 $in.^3$

$Z_{top\ CIP}$ = 483.0 / 4.72 = 102.3 $in.^3$

Check prestressing loss: (AASHTO Art. 9.16)

- Elastic shortening loss: (AASHTO Art. 9.162.1.2)

ES = $(E_s\, f_{cir} / E_{ci})$

E_s = 28,000 ksi

E_{ci} = 3,834 ksi

f_{cir} = stress at the center of gravity of prestressing force due to prestressing force and the self weight of the panel immediately after transfer

Assume 4% initial losses at release, therefore total prestressing force at release is:

P = (16 x 0.153) (202.5) (1-0.04) = 475.9 kips / panel width

f_{cir} = 475.9 / (4.5)(8x12) = 1.10 ksi

ES = $(E_s f_{cir} / E_{ci})$

= (28,000 x 1.10)/ 3,834= 8.03 ksi

- Shrinkage loss:

SH = 17,000 - 150(RH)

= [17,000 - 150(70)] / 1000 = 6.50 ksi

- Creep loss:

f_{cir} = 1.10 ksi

f_{cds} = concrete stress at the center of gravity of prestressing steel due to all dead loads except the dead load presented at the time of applying the prestressing force, i.e. due to CIP topping and wearing surface self loads. Since the CIP topping self weight is acting on the SIP panel, therefore the stresse resulting from it at the center of gravity of prestressing steel is zero.

Bending moment due to wearing surface = $0.025 \times 11.5^2 / 10 =$ 0.331 ft-kips

CIP top compression stresses = 0.331 x 12 / 102.3 = 0.039 ksi

Precast bottom tensile stresses = 0.331 x 12 / 127.8 = 0.031 ksi

From Fig. F9., concrete stress at the center of gravity of prestressing steel = 0.0125 ksi

CR_c = 12 f_{cir} - 7 f_{cds} = 12x1.10 - 7x0.0125 = 13.11 ksi

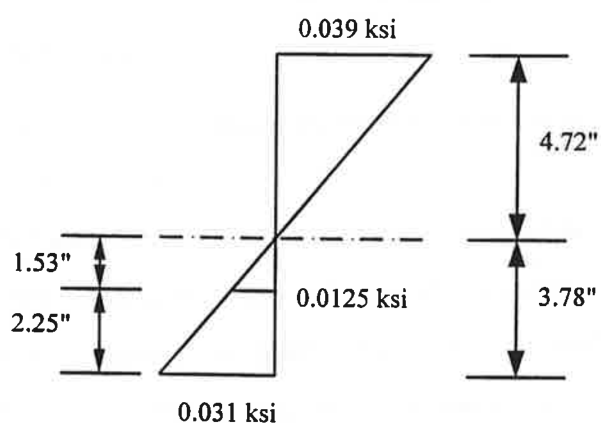


Fig. F-9. Concrete stresses

- Strand relaxation loss:

CR_s = 5.0 - 0.1(ES) - 0.05($SH+CR_c$) =

= 5.0 - 0.1x8.03- 0.05(6.5+13.11) = 3.22 ksi

- Initial loss (at release) = ES = 8.03 ksi

  Initial prestressing loss = 8.03 / 202.5 = 3.96 %

  ≅ Assumed initial losses (4%) OK

  Prestressing stress after initial loss = 202.5 - 8.03 = 194.47 ksi

  Total initial prestressing force = (16x0.153)(194.47) = 476.0 kips

- Final loss = 8.03 + 6.5 + 13.11 + 3.22 = 30.86 ksi

  Final prestressing loss = 30.86 / 202.5 = 15.2 %

  Final prestressing stress = 202.5 - 30.86 = 171.64 ksi

  Total final prestressing force = (16x0.153)(171.64) = 420.2 kips

Design of the SIP precast prestressed panel:

- Check concrete stresses of the precast panel at release:

Allowable stresses: (AASHTO Art. 9.15.2.2)

Compression = $0.6\ f_{ci}^{'}$

= 0.6 x 4.000 = 2.4 ksi

Tension with no bonded reinforcement = 200 psi or $3.0\sqrt{f_{ci}^{'}}$

= $3.0\sqrt{4000}$ / 1000 = 0.19 ksi < 0.20 ksi OK

Tension with bonded reinforcement = $7.5\sqrt{f_{ci}^{'}}$

= $7.5\sqrt{4000}$ / 1000 = 0.47 ksi

Total initial prestressing force, see previous section = 476 kips/panel

= 468 / (8) = 59.5 kips/ft

Since the strand pattern is symmetric, the precast panel will not have any camber. Therefore the self weight of the precast panel at release will not produce any stress through the panel. However, for convenience, an assumption is made such that the panel will be supported over the positions of girder lines shortly after release. Therefore the self weight of the precast panel is considered in the calculations of stresses at release.

Bending moment due to self weight of the panel $= 0.0563 \times 12^2 / 8 = 1.0134$ ft-kips

Concrete stresses at release at mid span are, (- value = compression, + value = tension):

Maximum compression $= -(59.5/54) - (1.0134 \times 12 / 40.5) =$

$= -1.083 - 0.300 \qquad = -1.383$ ksi

< -2.4 ksi OK

Minimum copression $= -(58.5/54) + (1.0134 \times 12 / 40.5) =$

$= -1.083 + 0.300 \qquad = -0.783$ ksi

< -2.4 ksi OK

- Check of stresses in strands and reinforcing bars at the gaps at release:

From previous section, total initial prestressing force is 476 kips/panel

Assume change in stress in the reinforcing strands is Δf_p , and change in stress in reinforcing bars is Δf_s.

From the compatibility equation:

$\Delta f_p/E_p = \Delta f_s/E_s$

$\Delta f_p = \Delta f_s\,(28{,}000/29{,}000) = 0.966\,\Delta f_s$ Eq. F-1

From equilibrium equation, total prestressing force is distributed between the prestressing strand areas, $A_p = 0.153 \times 16 = 2.448$ in^2, and the reinforcing bar areas, $A_s = 0.44 \times 28 = 12.32$ in^2.

$476 = \Delta f_s\,(A_s) + \Delta f_p\,(A_p)$

$= \Delta f_s\,(12.32) + \Delta f_p\,(2.448)$ Eq. F-2

From Eqs. 1 and 2: $\Delta f_s =$ 32.41 ksi , $\Delta f_p =$ 31.31 ksi

The prestressing strands has an intial stress of 194.47 ksi, see section 5, therefore, tension stress in the prestressing strands in the gap = 194.47 - 31.31 = 163.16 ksi

Compressive stress in the reinforcing bars in the gap = 32.41 ksi

Embedment length of the reinforcing bars:

The reinforcing bars in the gap must be adequate to satisfy two design criteria: (1) preserve as much prestress in the strands as possible, which is covered by section 6.2; and (2) transfer that prestress to the adjacent concrete without too much stress concentration. Although the reinforcing bars are expected to be predominantly in compression, a conservative approach is considered by using the tension development length specified by AASHTO Standard as the minimum required embedment into the concrete and by ignoring the end bearing of these bars. The first approach was considered to prevent the bars from slipping from the concrete in case of overloading the precast panel before and during casting the CIP topping. The second approach is considered to avoid stress concentration in the concrete by the end of the reinforcing bars.

If the bars are in tension: (AASHTO Art. 8.25.1)

For bars # 11 and smaller: $L_d = \dfrac{0.04A_b f_y}{\sqrt{f_c^{'}}} = \dfrac{0.04(0.44)(60{,}000)}{\sqrt{4{,}000}} = 16.7$ in.

but not less than $> (0.0004\ d_b\ f_y = 0.0004x0.75x60{,}000 = 18.0$ in.)

If the bars are in compression:

$$L_d = \frac{0.02d_b f_y}{\sqrt{f_c^{'}}} = \frac{0.02(0.75)(60{,}000)}{\sqrt{4{,}000}} = 14.23 \text{ in.}$$

but not less than $> (0.0003\ d_b\ f_y = 0.0003x0.75x60{,}000 = 13.50$ in.

therefore take L_d = 18.0 in.

Check buckling of the reinforcing bars at the gap:

Since the reinforcing bars in the gap are predominantly in compression, therefore buckling of these bars should be checked as follows *(Steel Structures Manual, Allowable Stress Design, Chapter E)*:

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2(29{,}000)}{60.0}} = 97.7$$ (AISC Eq. E2-1)

For #6 bar, $r = \sqrt{\dfrac{I}{A}} = \sqrt{\dfrac{\pi d^4 / 64}{\pi d^2 / 4}} = \dfrac{d}{4} = \dfrac{0.75}{4} = 0.1875$ in.

Effective slenderness ratio is Kl/r = (0.65x8)/(0.1875) $= 27.7 < C_c$

Allowable compression stress is : (AISC Eq. E2-1)

$$F_a = \frac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^2}\right]F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C_c} - \dfrac{(Kl/r)^3}{8C_c^3}} = \frac{\left[1 - \dfrac{(27.7)^2}{2(97.7)^2}\right](60.0)}{\dfrac{5}{3} + \dfrac{3(27.7)}{8x97.7} - \dfrac{(27.7)^3}{8(97.7)^3}} = 0.542(60.0) = 32.53 \text{ ksi}$$

From previous section, the compressive stress in the reinforcing bars in the gap is 32.41 ksi which is less than F_a.

Check handling stresses:

- Stresses in strands and reinforcing bars at the gaps after all losses occurs:

By the time the SIP panels are moved to the field, losses due to creep, shrinkage and relaxtion of prestressing steel will be already occured. Therefore, final prestressing force is 420.2 kips/panel, see check of pretressing losses section .

Using the same procedure used before in determining the stress at the gap, therefore:

$\Delta f_p = \Delta f_s\,(28{,}000/29{,}000) = 0.966\,\Delta f_s$ Eq. F-3

$420.2 \;= \Delta f_s\,(12.32) + \Delta f_p\,(2.448)$ Eq. F-4

From Eqs. 1 and 2: $\Delta f_s =$ 28.61 ksi , $\Delta f_p =$ 27.64 ksi

The prestressing strands has a final stress of 171.64 ksi, see section 5, therefore, tension stress in the prestressing strands in the gap = 171.64 - 27.64 = 144.00 ksi

Compressive stress in the reinforcing bars in the gap = 28.61 ksi

- Additional stresses during handling:

As stated in the design assumptions, that no loads should be provided on the overhangs of the SIP panels before the mortar mix filling of the gaps over the girder lines gets its specified strength, 4,000 psi, therefore only the self weight of the overhang of the SIP panel affects the stresses in the reinforcing bars. The length of the concrete part of the overhang is 44 inch, i.e 3.67 ft.

Bending moment at girder center line due to overhang self weight =

= (0.0563x3.67)(4- 0.5x3.67) = 0.45 ft-kips/ft

= 0.45 x 8 = 3.60 ft-kips/panel

The section at center line over the girder consists of 16-1/2" strands and 28-#6 bars.

Therefore, A_t = 16x0.153 + 28x0.44 = 2.448 + 12.32 = 14.768 in^2

$$I = (16\text{x}0.153)(1.0)^2 + \left[\frac{\pi(0.75)^4}{64} + 0.44(0.875)^2\right](28) =$$

$$= 2.448 + 9.867 = 12.315\ in^4$$

$$Z_{rebar} = 12.315/0.875 = 14.074\ in^3$$

$$Z_{strand} = 12.315/1.0 = 12.315\ in^3$$

Change of stresses in rebars = (3.6x12) / 14.074 = 3.07 ksi

Change of stresses in strands = (3.6x12) / 12.315 = 3.51 ksi

Maximum compresive stress in rebars = 28.61 + 3.07 = 31.68 ksi

From section 6.4, maximum allowable stresses in the reinforcing rebars, F_a, is 32.53 ksi. Therefore the stresses in the reinforcing bars are safe.

- Check shearing stresses in the gap over the girder lines:

Maximum shearing force = 0.0563x3.67 = 0.21 kips/ft

= 0.21 x 8 = 1.68 kips/panel

Actual shear stresses = 1.68/14.768 = 0.11 ksi

Allowable shear stress = 0.6 F_y = 0.6x60 = 36 ksi OK

Check stresses of the precast panel at time of casting top slab:

Assume that the SIP precast panel reaches the specified 28-days strength, 10,000 ksi, at the time of casting the topping concrete, therefore the allowable stresses of the precast prestressed panel are: (AASHTO Art. 9.15.2.2)

Compression: Under all load combinations $= 0.6\ f_c' = 0.6 \times 10.000 = 6.0$ ksi

Due to effective prestress plus permanent dead load

$= 0.4\ f_c' = 0.4 \times 10.000 = 4.0$ ksi

Due to live loads plus one half of the compressive stresses due to prestress and permanent dead loads

$= 0.4\ f_c' = 0.4 \times 10.000 = 4.0$ ksi

Tension: with bonded reinforcement $= 6.0\sqrt{10{,}000} / 1000 =$ 0.6 ksi

From section 5, final total prestressing force = 420.2 kips / panel

= 420.2 / 8.0 = 52.6 kips/ft

Three sections should be checked:

1. maximum positive moment section between the girders
2. maximum negative moment section of the overhang adjacent to the gap (prestressed concrete section)
3. maximum negative moment section of the overhang at the gap (reinforced concrete section)

4. Maximum positive moment section under to panel self weight, CIP deck topping self weight, and construction load is

$$= (0.0563+0.0563+0.05)\,(11.5^2) / 10 = 2.15 \text{ ft-kips}$$

Maximum compression stress on concrete $= -(52.6/54) - (2.15 \times 12/40.5)$

$= -0.974 - 0.637$

$= -1.611$ ksi < -6.0 ksi OK

Maximum tension stress on concrete $= -(52.6/54) + (2.15 \times 12/40.5)$

$= -0.963 + 0.637$

$= -0.337$ ksi < -6.0ksi OK

- Maximum negative moment section of the overhang adjacent to the gap (under panel self weight, CIP deck topping self weight, construction load and the reactions from the paving machine):

Bending moment due to SIP panel, CIP concrete and construction load =

$$= (0.0563 + 0.0563 + 0.050)(3.67^2)/2 = 1.10 \text{ ft-kips/ft}$$

From Fig. F-7., maximum reaction per wheel of the paving machine for 44 ft deck width

$$= 1234 + 60(44-30) = 2074 \text{ lbs.}$$

Only three reactions can be accommodated through the panel width, each reaction P is 2.074 kips. Therefore, bending moment is

$= (3 \times 2.074 \times 3.67) =$ 22.8 ft-kips/panel

$= 22.8 / 8.0 =$ 2.85 ft-kips

Total service moment $= 1.10 + 2.85 =$ 3.95 ft-kips

Maximum compression stress $= -(52.6/54) - (3.95 \times 12/40.5)$

$= -0.974 - 1.170$

$= -2.144$ ksi < -6.0 ksi OK

Maximum tension stress $= -(52.6/54) + (3.95 \times 12/40.5)$

$= -0.974 + 1.170$

$= +0.196$ ksi $< +0.6$ ksi OK

- Maximum negative moment section of the overhang at the gap (under panel self weight, CIP deck topping self weight, and the reactions from the paving machine):

As mentioned in the construction stages, a flowable mortar mix or grout is be used for a height of 2.25 inch from the bottom surface of the precast panel. The mix is assumed to have a 4,000 psi compressive strength at time of casting CIP topping. It helps to build the compression block against the applied loads and to provide bearing for the SIP panels over the girders.

Total factored moment = 1.3(1.10 + 1.67x2.85) = 7.30 ft-kips/ft

Note that the paving machine loads are considered as live loads.

Area of provided reinforcement = 2x0.44 + 1x0.153 = 1.033 in^2/ft

d = 4.5 - 1.25 = 3.25 in.

Neglecting the compression reinforcement, therefore:

$a = (A_s f_y)/(0.85bf'_c)$ = (1.033x60)/(0.85x12x4.0) = 1.52 in.

$\varphi M_n = \varphi A_s f_y (d - 0.5 a)$ = (0.9x1.033x60)(3.25-0.5x1.52) / 12

= 11.57 ft-kips/ft > 7.30 ft-kips/ft OK

Check allowable stresses at time of opening the bridge for traffic at maximum positive moment section:

At time of opening the bridge for traffic, the self weight of the wearing surface and the live load will act on the composite section. Therefore the allowable stresses are:

compression for CIP concrete $= 0.6\ f'_c = 0.6 \times 4.000$ = 2.4 ksi

for precast concrete $= 0.6\ f'_c = 0.6 \times 10.000$ = 6.0 ksi

tension for precast concrete $= 6.0\sqrt{10{,}000} / 1000$ = 0.6 ksi

Bending moment due to wearing surface load = $0.025 \times 11.5^2 / 10$ = 0.33 ft-kips/ft

Bending moment due to HS-25: (AASHTO Art. 3.24.3.1)

$M_{LL+I} = 0.8\ [\ P_{25}\ (S+2) / 32\ ]\ (1+I) = 0.8\ [\ 20\ (11.5+2) / 32\ ]\ (1+0.3)$

= 8.78 ft-kips /ft

Therefore, stresses due to composite loads are (-value = compression):

Compression stresses at top of CIP concrete = -n(0.33+8.78)(12)/ 102.3

= (0.63)(-1.068) = 0.673 ksi < 2.4 ksi OK

Tension stresses at top of precast panel = -(0.33+8.78)(12)/ 670.8 = -0.163 ksi

Tension stresses at bottom of precast panel = +(0.33+8.78)(12)/ 127.8 = +0.855 ksi

In order to get final stresses on the precast panel, add the stresses due to the composite loads (wearing surface and live load) to those calculated in section (6.5).

Final stress at top of precast panel = -1.611 - 0.163 = -1.774 ksi (comp.)

< 6.00 ksi OK

Final stress at bottom of precast panel = - 0.337 + 0.855 = +0.518 ksi (tension)

< 0.60 ksi OK

Check ultimate capacity of the composite section at maximum positive moment section:

(AASHTO Art. 9.17)

M_u = 1.3 [M_D + 1.67 M_{L+I}]

= 1.3 [(0.0563+0.0563+0.025)(11.5^2)/10 +1.67(8.78)]

= 21.4 ft-kips/ft

A_s = (16 x 0.153) / (8) = 0.306 in.^{2}/ft

d = 4.0 + 2.25 = 6.25 in.

ρ = 0.306 / (6.25 x 12) = 0.00408

f_c' (CIP concrete) = 4.0 ksi

f_s' = 270 ksi

$$f^*_{su} = f_s'\left(1 - \frac{\gamma^*}{\beta_1}\frac{\rho f_s'}{f_c'}\right) \qquad \text{(AASHTO Eq. 9-17)}$$

$$= 270\left(1 - \frac{0.28}{0.85} \times \frac{0.00408 \times 270}{4.0}\right) = 245.5 \text{ ksi}$$

Check f^*_{su} : (AASHTO Eq. 9-19)

$f^*_{su} \leq (L_x/D + .667\, f_{se})$

L_x = 11.5 / 2 = 5.75 ft , D = 0.5 in. , f_{se} = 171.64 ksi

(L_x/ϕ + 0.667 f_{se}) = (5.75x12/0.5 + 0.667x171.64) = 252.5 ksi > f^*_{su} OK

Therefore, f^*_{su} = 236 ksi controls

φM_n = $\varphi A_s f^*_{su} d$ (1 - 0.6 $\rho f^*_{su} / f_c'$) (AASHTO Eq. 9-13)

= 1.0 x 0.306 x 245.5 x 6.25 (1 - 0.6 x 0.00408 x 245.5 / 4.0)/12

= 33.24 ft-kips /ft > M_u = 21.4 ft-kips OK

Check ductility limits: (AASHTO Art. 9.18)

- Check maximum prestressing steel: (AASHTO Art. 9.18.1)

$(\rho f^{*}_{su} / f'_c) = (0.00378 \text{ x } 236 / 4.0) = 0.223 \leq \quad (0.36\beta_1 = 0.36\text{x}0.65 = 0.234)$

OK

- Check minimum steel: (AASHTO Art. 9.18.2)

$1.2\, M_{cr} \leq \varphi M_n$

$$M_{cr} = (7.5\sqrt{f'_c} + \frac{P}{A_{non-comp.}} + \frac{Pe}{S_{non-comp.}})S_{bott.Comp} - (M_{panel\ wt.} + M_{CIPslab})(\frac{S_{bott.comp.}}{S_{bott.non-comp.}} - 1)$$

P $= 420.2/8.0 = 52.6$ kips

$$M_{cr} = (7.5\frac{\sqrt{10{,}000}}{1{,}000} + \frac{52.6}{54} + 0.0)(127.8) - (0.745 + 0.745)(12)(\frac{127.8}{40.5} - 1) =$$

$= 181.8$ kip-in.$=$ 15.14 ft-kips

$1.2\, M_{cr} = 18.18$ ft-kips $\leq \varphi M_n$ OK

Distribution reinforcement steel in the precast prestressed panel:

(AASHTO Art. 9.23.2)

$A_{S\ min.} = 0.11$ in^2./ ft

#4 bar at the pockets provides 0.10 in^2/ft which is 91% of the required $A_{S\ min.}$. The research team decided to use #4 bar because larger size of bars requires more development length which is not provided with the optomized dimensions of the pockets. See section 8 of the calculations. However, to satisfy the AASHTO requirements with the available dimensions of the pockets, the spacing between the pockets may be reduced to 1.5 ft. Finally, the research team decided to go with # 4 bars spaced at 2 ft.

Design of section of negative moment over interior girder lines

- Bending moment:

Negative bending moment over the interior girder lines is provided by the self weight of the waering surface and the live loads.

DL: Due to wearing surface, $M_{ws} = 0.025\ (11.5^2/10) = 0.331$ ft-kips

LL: Impact: $I = 30\%$ (AASHTO Eq. 3-1)

For HS-20, $M_{LL} = \left(\frac{S+2}{32}\right)P_{20}$ (AASHTO Eq. 3-15)

Thus, for HS-25, 0.30 impact, and 0.8 continuity factor (AASHTO Art. 3.24.3.1):

$M_{LL} = 1.25\left(\frac{11.5+2}{32}\right)(16)(1+0.30)(0.8) =$ 8.775 ft-kips

Total service bending moment $= 0.331 + 8.775 =$ 9.106 ft-kips

Total factored bending moment, $M_u = 1.3(0.331 + 1.67 \text{x} 8.775) =$ 19.481 ft-kips

- Design of section:

Neglect the effect of the prestressing strands and #6 bars running in the gap over the girder line.

Assume using D31, thus with 2.5 in. clear cover, therefore:

$d = 9 - 0.5 \text{x} 0.628 - 2.5 =$ 6.186 in.

$b = \quad =$ 12.0 in.

$R_n = (M_u / \phi b d^2) =$ 565.7 lb/in^2, $(\phi = 0.9)$

$m = (f_y / 0.85 f_c') =$ 17.65

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - 2\frac{mR_n}{f_y}}\right) = 10.38 \times 10^{-3}$$

$A_s = \rho\,(bd) =$ 0.771 in^2

Use W31 @ 4 in., $A_s =$ 0.93 in^2

$a = (A_s f_y) / (0.85\, b\, f_c') =$ 1.37 in.

$\varphi M_n = 0.9\,(A_s f_y)(d - 0.5a) =$ 23.022 ft-kips > M_u OK

- Check minimum reinforcement: (AASHTO Art. 8.17.1)

$1.2M_{cr} = 1.2(1.25\, bH^2 \sqrt{f_c'}) = 1.2 \times 1.25 \times 12 \times 9^2 \sqrt{4000} / (12{,}000) = 7.6$ ft-kips

$< \phi M_n$ OK

- Check crack control for distribution of reinforcement: (AASHTO Art. 8.16.8.4)

$d_c = (2.5 + 0.5 \times 0.628) =$ 2.814 in.

$A_c = (2 \times 2.814 \times 3) =$ 16.88 in^2

$Z =$ 170 kip/in.

$$f_{sa} = \frac{Z}{\sqrt[3]{A_c d_c}} = 47 \text{ ksi} > (0.6 f_y = 36 \text{ ksi})$$

$f_{sa\ all.} =$ 36 ksi

Actual stress in reinforcement:

E_c = $33\,(w_c)^{1.5}\sqrt{f'_c}$ = 3835 ksi

n = E_s / E_c = 7.56

ρ_{act} = 1.24 / (12x6.186) = 16.70×10^{-3}

k = $\sqrt{(\rho n)^2 + (2\rho n)} - \rho n$ = 0.52

j = $(1 - k/3)$ = 0.83

$f_{sa\ act.}$ = $M_{service} / (jdA_s)$ = 22.9 ksi < $f_{sa\ all.}$

OK

Design of section of negative moment over exterior girder line:

- Bending moment:

Assume wheel load applied at 1 ft from face of railing, as shown in Fig. F-10.

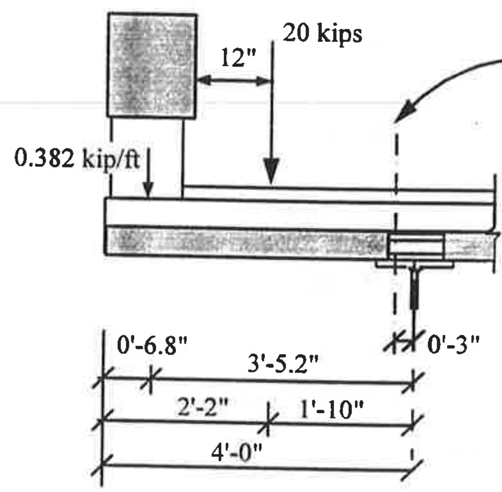


Fig. F-10. HS-25 Loading for the Overhang

$M_{D\,(slab+ws)}$ = (0.025) (3.75^2) / 2 = 0.176 ft-kips

$M_{D\,(barrier)}$ = 0.382 (3.433-0.25) = 1.216 ft-kips

Live load: wheel load is distributed over a width of (AASHTO Art. 3.24.5)

E = 0.8 X + 3.75 = 0.8(1.833-0.25) + 3.75 = 5.02 ft

M_{L+I} = 1.30 x 20 x (1.833-0.25) / 5.02 = 8.199 ft-kips

Total service bending moment = 0.176 + 8.199 = 8.375 ft-kips

Total factored bending moment = 1.3(0.176 + 1.67x8.199)

= 18.028 ft-kips

These values are less than those used to design negative moment section over interior girder lines. Thus, same reinforcement used for sections over interior girder lines can be used here.

Check two way shear (punching shear): (AASHTO Art. 3.30)

Tire contact area = 0.01 P = 0.01 x 20,000

= 200 in^2 = b x w

and w = 2.5 b

Therefore, b = 8.94 in. , w = 22.36 in.

d (at positive moment section) = 4.5 + 4.0 - 1.25 = 7.25 in.

b_o = 2 [(b+d) + (w+d)] = 90.60 in.

$$V_{c\ provided} = (0.8 + \frac{2}{2.5})\sqrt{f'_c}\ b_o d =$$

$$= (0.8 + \frac{2}{2.5})\sqrt{4000}\ (91.60x7.25) / 1000 \quad = \quad 67.2 \text{ kips}$$

V_u = (Wheel load x Impact x Load factor)

= 20 x 1.3 x (1.3x1.67) = 56.45 kips $< V_{c\ provided}$ OK

Distribution reinforcement: (AASHTO Art. 8.20)

$A_{s\ min.}$ = (1/8) in.^{2}/ ft = 0.125 in.^{2}/ ft

Use D7 @ 6", A_s = 0.14 in.^{2}/ ft > $A_{s\ min.}$ OK

Check development length of the # 4 bars at the pockets

If the bars are in tension: (AASHTO Art. 8.25.1)

For bars # 11 and smaller: $L_d = \frac{0.04 A_b f_y}{\sqrt{f'_c}}$

$$= \frac{0.04(0.20)(60,000)}{\sqrt{4,000}} = 7.6 \text{ in.}$$

but not less than $(0.0004\ d_b\ f_y)$ = 0.0004x0.50x60,000 = 12.0 in. Controls

For reinforcement enclosed within a spiral of not less than 1/4 inch in diameter and not more than 4 inch pitch, the development length can be reduced by 0.25 (Art. 8.25.3.3)

Therefore, L_d = 12 x 0.75 = 9 inch

Note that AASHTO Standards does not account for the effect of using a pitch less than 4 inch. However the maximum available splice length is 4.5 in. which is not consistant with the AASHTO specifications. Research recently done at the University of Nebraska showed that 4.5 in. is enough to fully develop #4 bars confined with 3 in. O.D., 2.5 in. I.D., 1 in. pitch, spirals.

Full-Depth Precast Deck System

The configuration of the proposed bridge is shown in Fig. F-11.

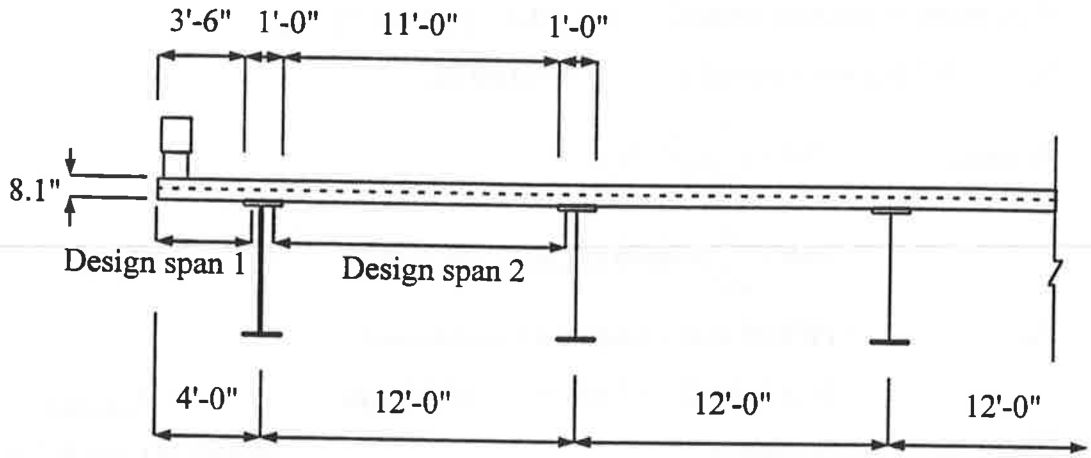


Fig. F-11. Design Panel Configuration

Design Conditions

Design calculation follows the *AASHTO Standard Specifications for Highway Bridges.*

Live Load	HS 25
Girder Spacing	12'
Concrete Strength	7500 psi

Design Spans

$$\text{Design span 1: } l_s = 3.5' + \frac{b}{4} \approx 3.75'$$

$$\text{Design span 2: } l_s = 11' + \frac{b}{2} = 11.5$$

Section Properties

Section properties of the precast panel will be calculated based on the cross-section as shown in Fig. F-12.

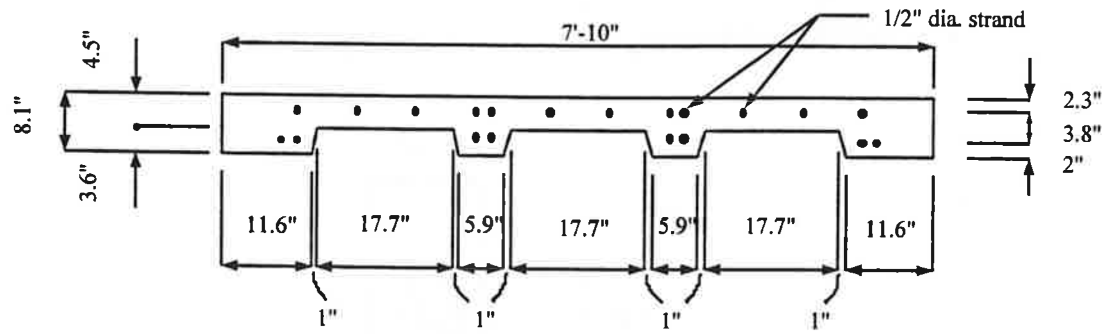


Figure F-12. Design Section

Positive moment zone

$$A_g = (94"\times4.5") + (11.6"\times2 + 5.9"\times2)\times3.6" + (\frac{1}{2}\times1"\times3.6")\times6$$

$$= 423 + 126 + 11 \qquad = 560\ \text{in}^2$$

$$Ay = 423\times5.85" + 126\times1.8" + 11\times2.4" = 2728\ \text{in}^3$$

$$y_{top} = 8.1" - \frac{2728}{560} = 3.23\ \text{in}, \qquad y_{bot} = -\frac{2728}{560} = -4.87\ \text{in}$$

$$I_g = 423\times(5.85"-4.87)^2 + \frac{94"\times(4.5")^3}{12}$$

$$126\times(1.8-4.87)^2 + \frac{35"\times(3.6")^3}{12}$$

$$11\times(2.4-4.87)^2 + 6\times\frac{1"\times(3.6")^3}{36} \qquad = 2519\ \text{in}^4$$

$$S_{top} = \frac{2519}{3.23} = 780\ \text{in}^3, \qquad S_{bot} = \frac{2519}{-4.87} = -517\ \text{in}^3$$

Negative moment zone

$$A_g = 94\times8.1 \qquad = 761\ \text{in}^2$$

$$y_{top} = -y_{bot} \qquad = 4.05\ \text{in}$$

$$I_g = 94\times(8.1)^3/12 = 4163\ \text{in}^4$$

$$S_{top} = -S_{bot} = \frac{4163}{4.05} = 1028\ \text{in}^3$$

Design Moments

Dead loads:

Self Weight (Slab + 2" thick wearing surface)

$$A_g = (\frac{560\ in^2}{8'} + 2" \times \frac{12"}{1'}) \times \frac{1}{12^2} \qquad = 0.653\ ft^2 / ft$$

$$w = 0.653\ {}^{ft^2}\!/_{ft} \times 0.150\ {}^{kips}\!/_{ft^3} \qquad = 0.098\ kip / ft$$

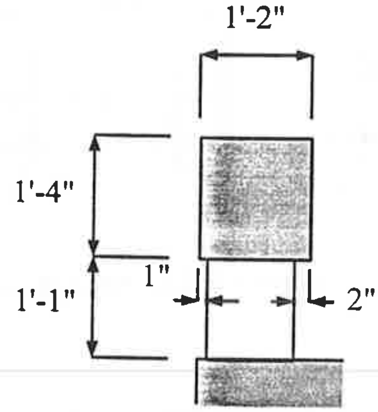


Fig. F-13. Solid Concrete Rail

$$A = 2.546\ ft^2,\ x_0 = 0.568\ ft,\ w = 0.382\ {}^{kips}\!/_{ft}$$

Span Moment (Positive Moment):

Dead Load

$$M_D = \frac{wl^2}{10} = \frac{(0.098)(11.5)^2}{(10)} = 1.30\ ft - kips / ft$$

Live Load (HS 25)

$$M_L = \frac{l+2}{32} P(0.8) \ldots\ldots\ldots (AASHTO\ Eq.\ 3-15)$$

$$P = 50\,kips \times 0.4 = 20\ kips$$

$$Impact = 25\ \%\ (Assumed)$$

$$M_{L+I} = \frac{11.5+2}{32}(20)(0.8)(1.25) \quad = 8.44\ ft - kips / ft$$

Cantilever Moment (negative Moment):

Dead Load

$$M_D = \frac{wl^2}{2} + pl$$

$$= \frac{(0.098)(3.57)^2}{2} + (0.38)(3.18)$$

$$= 1.83 \text{ ft} - \text{kips/ft}$$

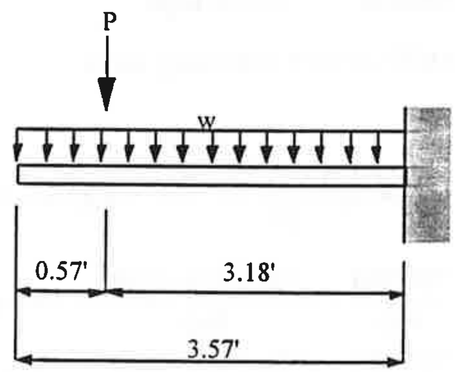


Fig. F-14. Loading Condition for Overhanging

Live Load (Article 3.24.5)

$E = 0.8x + 3.75$ ....(AASHTO Eq. 3 – 17)

Wheel load 1.0' from curb

$x = 3.57 - 1.167 - 1.0 = 1.40$ ft

$E = 0.8(1.40) + 3.75 \qquad = 4.87$ ft

$M_{L+I} = \frac{P}{E}x = \frac{20}{4.87}(1.40)(1.25) \qquad = 7.19$ ft – kips / ft

Negative moment at interior girders controls

$M_{L+I} = 8.44$ ft – kips / ft

Rail Collision Force

$E = 0.8x + 3.75$

$= 0.8(3.18) + 3.75 \qquad = 6.294$

x: distance from the center of the post to the point under investigation.

height of rail = 2.42 ft

$M_C = \frac{(20/2)}{6.294}(2.42) \qquad = 3.84$ ft – kips / ft

Prestressing force and prestress

Use 270k 1/2" diameter strands

Prestressing force

$$f_{se} = 0.75f'_S \eta$$

$$\eta = \frac{P_e}{P_i} = 0.85 \text{ (Assumed)}$$

$$f_{se} = 0.75(270)(0.85) \qquad = 172 \text{ ksi}$$

$$P_e = f_{se}A_p = (172)(0.153) \quad = 26.3 \text{ kips / strand}$$

$$\Sigma P_e = 26.3 \times 20 \text{ strands} \qquad = 526 \text{ kips}$$

Concrete stresses due to effective prestressing force

Positive moment zone:

$$\text{Eccentricity: } e = \frac{(8 \times 2") + (12 \times 5.8")}{20} - 4.87 = -0.59 \text{ in}$$

$$f_{top,pre} = \frac{526000}{560} + \frac{(526000)(-0.59)}{780} = 541 \text{ psi}$$

$$f_{bot,pre} = \frac{526000}{560} + \frac{(526000)(-0.59)}{-517} = 1540 \text{ psi}$$

Negative moment zone:

$$\text{Eccentricity: } e = \frac{(8 \times 2") + (12 \times 5.8")}{20} - 4.05 = 0.23 \text{ in}$$

$$f_{top,pre} = \frac{526000}{761} + \frac{(526000)(0.23)}{1028} = 809 \text{ psi}$$

$$f_{bot,pre} = \frac{526000}{761} + \frac{(526000)(0.23)}{-1028} = 574 \text{ psi}$$

Stresses during handling

Since eccentricity of prestressing steel is small, precast panel will experience negligible camber after release. The most critical state of the precast panel occurs during handling (See Fig. F-15).

Self weight: $w = A_{precast} \cdot 0.150 \text{ kcf} = (0.560)(0.150) = 0.084 \text{ kips / ft}^2$

$$M_A = M_B = -\frac{1}{2}(0.084)(14)^2 \qquad = -8.23 \text{ ft - kips / panel}$$

$$M_C = -\frac{1}{2}(0.084)(28)^2 + (2.35)(14) \quad = \quad 0 \text{ ft - kips / panel}$$

These moments are small enough to waive a stress check

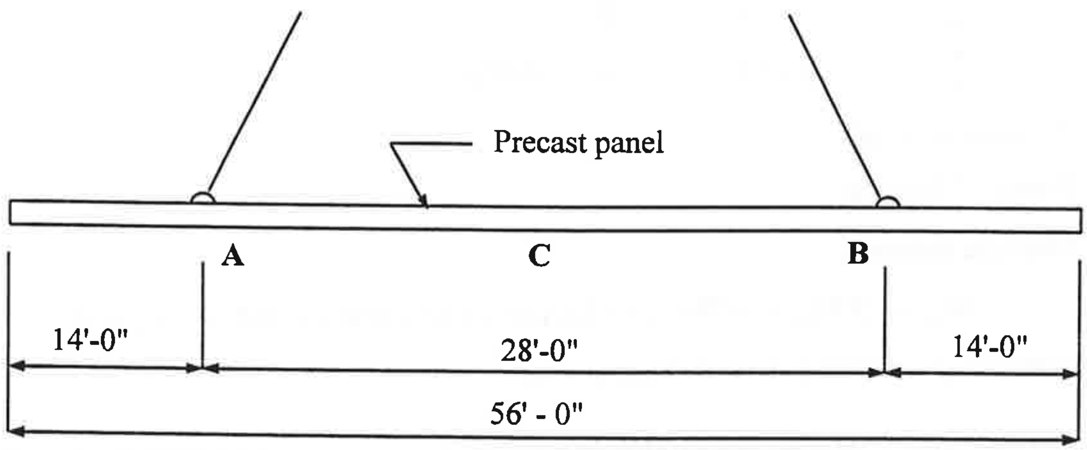


Fig. F-15. Precast Panel During Handling

Stresses due to service loads and total stresses

Span Moment (Positive Moment)

Design moment due to service load

$$M_{tot} = M_D + M_L = 1.30 + 8.44 = 9.74 \ \text{ft} - \text{kips/ft}$$

$$f_{top} = \frac{M_{tot}}{S_{top}} = \frac{(9.74 \times 12000 \times 8\,\text{ft})}{780} = 1199 \ \text{psi}$$

$$f_{bot} = \frac{\cancel{M}_{tot}}{S_{bot}} = \frac{(9.74 \times 12000 \times 8\,\text{ft})}{-517} = -1809 \ \text{psi}$$

Total Stresses

$$\Sigma f_{top} = 1199 + 541 \quad = 1740\,\text{psi} \quad < 0.6f'_C \quad = 4500\,\text{psi}$$

$$\Sigma f_{bot} = -1809 + 1540 \quad = -269\,\text{psi} \quad > -6\sqrt{f'_C} = -520\,\text{psi}$$

Cantilever Moment (Negative moment)

Design moment due to service load

$$M_{tot} = M_D + M_L = 1.83 + 8.44 = 10.27 \ \text{ft} - \text{kips/ft}$$

$$f_{top} = \frac{\cancel{M}_{tot}}{S_{top}} = \frac{(-10.27 \times 12{,}000 \times 8\text{ft})}{1028} = -959\,\text{psi}$$

$$f_{bot} = \frac{\cancel{M}_{tot}}{S_{bot}} = \frac{(-10.27 \times 12{,}000 \times 8\text{ft})}{-1028} = 959\,\text{psi}$$

Total Stresses

$$\sum f_{top} = -959 + 809 \quad = -150\,psi > -520\,psi$$

$$\sum f_{bot} = 959 + 574 = 1533\,psi > 4500\,psi$$

Flexural Strength

Positive Moment:

Ultimate moment

$$M_u = 1.3(M_D + 1.67M_{L+I}) = 1.3(1.30 + 1.67 \times 8.44) = 20.0 \ ft - kips/ft$$

Stress in prestressing steel at ultimate load

$$f_{SU} = f'_S[1 - (\frac{\gamma}{\beta_1})(\rho\frac{f'S}{f'_C})] \text{ ....(AASHTO Eq. 9 – 17)}$$

$$A_S = \frac{(0.153\,in^2)(8\,strands)}{8\,ft} = 0.153\,in^2$$

$$d = 6.1"$$

$$\rho = \frac{A_S}{bd} = \frac{0.153}{(12")(6.1")} = 0.00209$$

$$f'_C = 7500\,psi, \quad f'_S = 270\,ksi$$

$$\gamma = 0.28 \ \text{(Article 9.1.1)}$$

$$\beta_1 = 0.675 \ \text{(Article 8.16.2.7)}$$

$$f_{SU} = (270)[1 - (\frac{0.28}{0.675})(0.00209 \times \frac{270}{7.5})] = 262\,ksi$$

Nominal strength

$$M_n = A_S f_{SU} d(1 - 0.6\frac{\rho f_{SU}}{f'_C}) \text{ ....(AASHTO Eq. 9 – 13)}$$

$$= (0.153)(262)(6.1)[1 - 0.6\frac{(0.00209)(262)}{7.5}] = 234\,ft - kips/ft$$

$$\phi M_n = (1.0)(234) \times \frac{1}{12"} = 19.5\,ft - kips/ft \quad \approx \quad M_u = 20.0 \ ft - kips/ft$$

Minimum Prestressing Steel (Article 9.18.1)

$$\frac{\rho \cdot f_{SU}}{f'_C} = \frac{(0.00209)(262)}{(7.5)} = 0.073 \quad < \quad 0.36\beta_1 = 0.24$$

Negative Moment:

Ultimate moment

$$M_u = 1.3(M_D + 1.67M_{L+I}) = 1.3(1.83 + 1.67 \times 8.44) = 20.7 \text{ ft} - \text{kips / ft}$$

Stress in prestressing steel at ultimate load

$$f_{SU} = f'_S [1 - (\frac{\gamma}{\beta_1})(\rho \frac{f'_S}{f'_C})] \text{ .... (AASHTO Eq. 9 – 17)}$$

$$A_S = (0.153 \text{in}^2)(12 \text{ strands/stem}) = 0.230 \text{in}^2$$

$$d = 5.8"$$

$$\rho = \frac{A_S}{bd} = \frac{0.230}{(12")(5.8")} = 0.00330$$

$$f'_C = 7500 \text{psi}$$

$$f'_S = 270 \text{ksi}$$

$$\gamma = 0.28 \text{ (Article 9.1.1)}$$

$$\beta_1 = 0.675 \text{ (Article 8.16.2.7)}$$

$$f_{SU} = (270)[1 - (\frac{0.28}{0.675})(0.00330 \times \frac{270}{6})] = 253 \text{ksi}$$

Nominal strength

$$M_n = A_S f_{SU} d(1 - 0.6\frac{\rho f_{SU}}{f'_C}) \text{ .... (AASHTO Eq. 9 – 13)}$$

$$= (0.230)(253)(5.8)[1 - 0.6\frac{(0.00314)(253)}{7.5}] = 316 \text{ kips-in/ft}$$

$$\phi M_n = (1.0)(316) \times \frac{1}{12"} = 26.3 \text{ ft} - \text{kips / ft} \quad > \quad M_u = 20.7 \text{ ft} - \text{kips / ft}$$

Minimum Prestressing Steel (Article 9.18.1)

$$\frac{\rho \cdot f_{SU}}{f'_C} = \frac{(0.00330)(253)}{(7.5)} = 0.111 \quad < \quad 0.36\beta_1 = 0.24$$

Distribution reinforcement

For main reinforcement perpendicular to traffic (Article 3.24.10.2).

$$\frac{220}{\sqrt{S}} = \frac{220}{\sqrt{11.5}} = 64.9 \text{ \%} \quad < 67 \text{ \% maximum}$$

Required steel area: $A_{s,req} = \frac{(0.153)(8 \text{ strands})(0.65\%)}{8 \text{ ft}} = 0.099 \text{ in}^2 \text{ / ft}$

Use welded wire Fabric W5.5 x W5.5 x 6 x 6

$$A_s = 0.049 \times \frac{12''}{6''} = 0.099\ \text{in}^2/\text{ft} \geq A_{s,req}$$

Longitudinal reinforcement in negative moment region of continuous bridge

Since longitudinal post-tensioning is applied, this requirement is waived. Efficiency of the longitudinal post-tensioning will be checked by finite element analysis and experimental test.

Design for Shear

Since stems of this section are very shallow, the panel would behave as a slab. Therefore, one way shear is ignored and only punching (two way) shear is checked.

$$A = 0.01P = 0.01(20{,}000\,\text{lb}) = 200\,\text{in}^2$$

$$2.5b^2 = 200\,\text{in}^2$$

$$b = 8.94\,\text{in}$$

$$w = 2.5 \times 8.94 = 22.35\,\text{in}$$

$$t = 4.5\,\text{in}, \quad d = 3.0\,\text{in}$$

$$b_0 = 2(25.4 + 11.9) = 74.6\,\text{in}$$

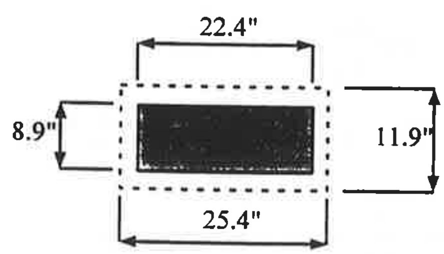


Fig. F-16. Tire Contact Area (Article 3.30)

Stress Design

$$v_c = \frac{V}{b_0 d} \cdots\cdots\cdots (\text{AASHTO Eq. } 8-12)$$

$$= \frac{(20{,}000 \times 1.25)}{(74.6)(3.0)} = 112\,\text{psi}$$

$$V_C = (0.8 + \frac{2}{\beta_C})\sqrt{f'_C} \quad \text{or} \quad 1.8\sqrt{f'_C}$$

O.K.

$$= (0.8 + \frac{2}{2.5})\sqrt{7500} = 139\ \text{psi} \quad > \quad v = 112\ \text{psi}$$

$$1.8\sqrt{7500} = 156\ \text{psi}$$

Strength Design

$$V_u = 1.3(1.67P) = 1.3(1.67 \times 20 \times 1.25) = 54.3 \text{ kips}$$

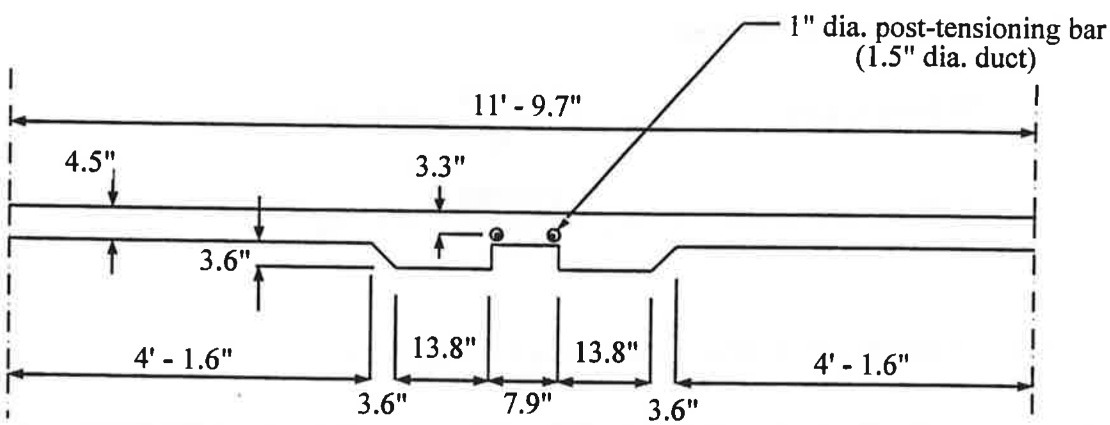
$$V_n = (2 + \frac{4}{\beta_C})\sqrt{f'_C}\,b_0 d \quad \text{or} \quad 4\sqrt{f'_C}\,b_0 d \text{ ....(AASHTO Eq. 8 - 58)}$$

$$= (2 + \frac{4}{2.5})\sqrt{7500}(74.6)(3.0) = 69.8 \text{ kips} \;>\; V_u = 54.3 \text{ kips}$$

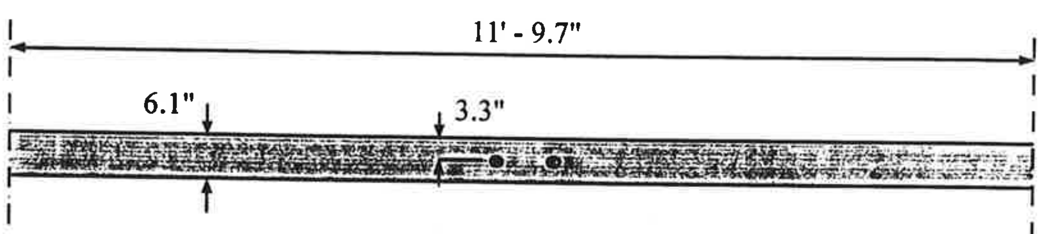
$$4\sqrt{7500}(74.6)(3.0) = 77.5 \text{ kips}$$

Longitudinal Post-tensioning

Compressive prestress of 200 psi shall be applied to transverse cross-section. Design sections are shown in Fig. F-17.



At Precast Panel



At Transverse Joint

Fig. F-17. Design Sections for Longitudinal Post-tensioning

Section properties:

At Precast panel:

$$A = 751 \text{ in}^2$$
$$y_t = 2.84 \text{ in}$$
$$e = 2.84 - 3.3 = -0.5 \text{ in}$$

At Transverse Joint:

$$A = 866 \text{ in}^2$$
$$y_t = 3.05 \text{ in}$$
$$e = 3.05 - 3.3 = -0.25 \text{ in}$$

Post-tensioning force

Use two 150 ksi threaded bars.

$$P_e = 0.6f_{pu}A_{ps} = 0.6 \times 150 \times 2 \times 0.85 = 153 \text{ ksi}$$

Compressive stress in concrete

At precast panel: $$f_{c,precast} = \frac{153{,}000}{751} = 204 \text{ psi}$$

At Transverse joint:: $$f_{c,precast} = \frac{153{,}000}{866} = 177 \text{ psi}$$

Results of design calculation is summarised in Fig. F-18.

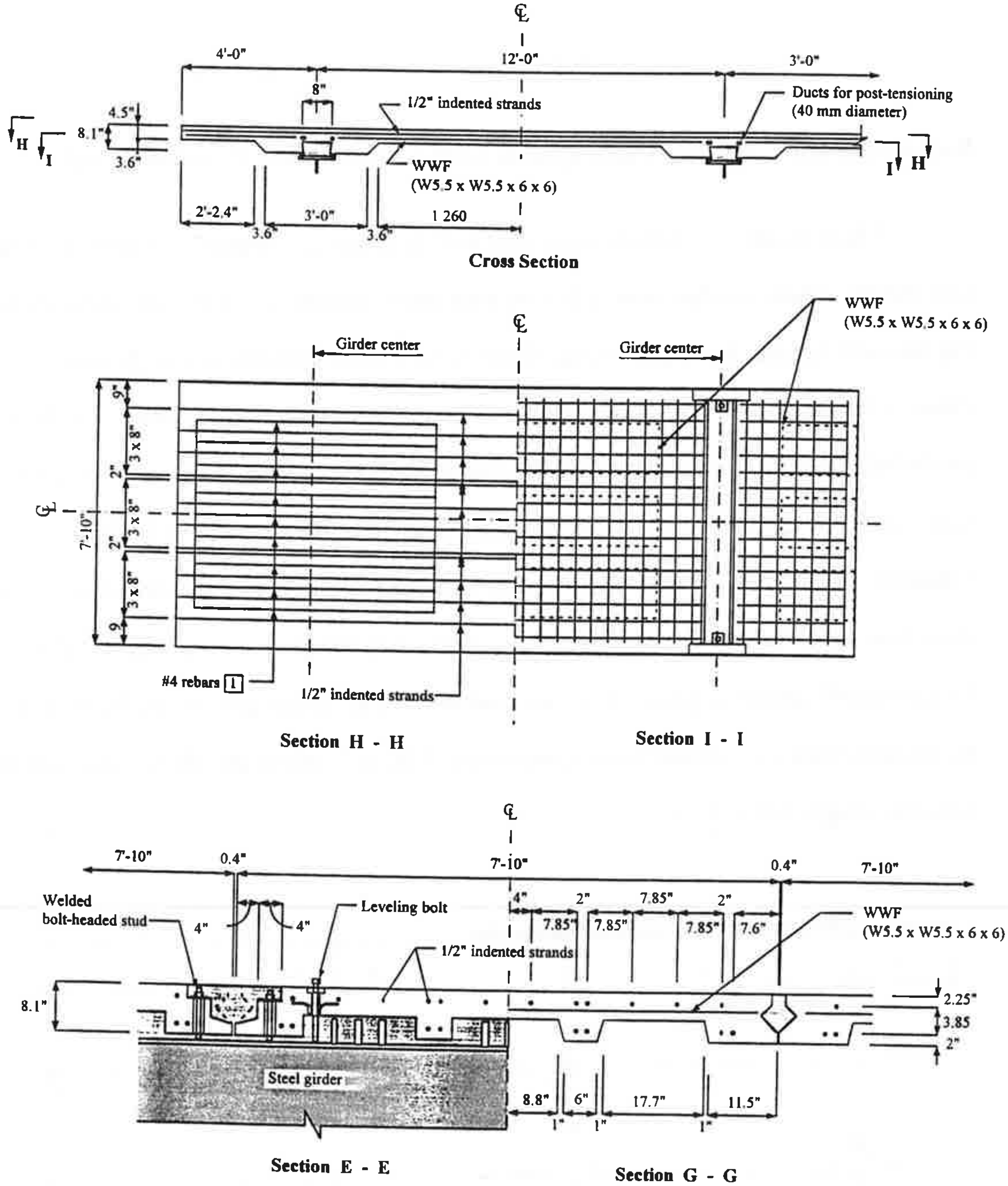


Fig. F-18. Reinforcing Aangement for Precast Panel